# DESIGN OF LOW-COST FLOWTHROUGH ROCKFILL DAMS AND SPILLWAYS

RENEWABLE ENERGY BRANCH

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## Preface

This report consists of a preliminary investigation of the hydraulic characteristics of flowthrough rockfill dams for low-cost small hydro projects. The study described in this report relates to the objective of Energy, Mines and Resources Canada to incorporate new technologies in the design of facilities to harness renewable sources of energy. Flowthrough rockfill dams are perceived to be an attractive alternative for the design of low head hydro schemes because they may reduce the costs of civil works for such projects.

This report is intended for civil engineers involved in the design of low head hydro dams. It is emphasised that the promising conclusions presented in this work are of a preliminary nature and will require detailed verification in the subsequent phase of this study.

Abstract

A literature review was performed on the present knowledge of non-Darcy flow through porous media, of seepage through rockfill, and of the hydraulic design of flowthrough rockfill dams. The literature review showed that power-type expressions for non-Darcy flow through porous media vary greatly as to values of the empirical coefficient in these expressions. The literature review also showed that traditional design methods for rockfill dams of the type considered in this report are divided into overflow design and flowthrough design.

A preliminary parametric study was performed of the flowthrough characteristics of rockfill placed in a test cylinder 1 m long and 2β.5 cm in diameter. The influence of such factors as rock size, shape, gradation, degree of compaction, and angularity were investigated. It was found that for crushed limestone the coefficient in a power-type expression for non-Darcy flow can be estimated if the mean hydraulic radius is known. It is therefore possible that a governing equation for non-Darcy flow be obtained without performing expensive hydraulic experiments. More experiments are however needed to further substantiate results presented in this study.

Preliminary tests in a flume 13 m long and 38 cm wide were performed to study the hydraulic behaviour of model flowthrough rockfill dams. It was found that a model dam having an impervious element (cut-off wall) extending only part way to the crest did not demonstrate greater hydraulic stability than a model dam having a cut-off wall extending the full height of the model dam. Use of partial cut-off walls, which do not maximise the available head for a given volume of fill, do not therefore represent the most economical design. The flume studies also indicate that it may be desireable to unify the concepts of 'overflow' and 'flowthrough' in rockfill dams for the hydraulic design of such structures, since significant flowthrough occurs in 'overflow' structures.

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Executive Summary

Flowthrough rockfill dams are perceived to be an attractive alternative for the design of low head hydro schemes because they may reduce the costs of civil works for such projects.

A literature review was performed on the present knowledge of non-Darcy flow through porous media, of seepage through rockfill, and of the hydraulic design of flowthrough rockfill dams. Although existing research on non-Darcy flow in porous media is extensive, much of this work is presented in the literature in a form which is not useful for non-Darcy seepage analysis, that of modified Moody diagrams. The literature review indicated that power-type expressions for non-Darcy flow through porous media vary greatly as to the values of the empirical parameters in these expressions. This variation is probably due to differences in the types of porous media used in previous studies. Certain key parameters, such as the crest head and the critical exit height have been found to determine important aspects of the behaviour of rockfill dams having inbuilt spillways. It was also found that existing design methods for rockfill dams can be divided into 'overflow' design and flowthrough design methods.

A preliminary parametric study was performed of the flowthrough characteristics of rockfill placed in a test cylinder 1 m long and 28.5 cm in diameter. The influence of such factors as rock size, shape, gradation, degree of compaction, and angularity were investigated. For crushed limestone ranging in size from about 16 mm to 36 mm it was found that the coefficient in a power-type expression for non-Darcy flow can be estimated if the mean hydraulic radius of the porous media is known. This latter parameter was determined knowing the mass-specific surface area for the rock particles, which was estimated using a special coating technique.

Preliminary tests in a glass-walled flume 13 m long and 38 cm wide were performed to study the hydraulic behaviour of model flowthrough rockfill dams. Tests were performed to check the reproducibility of stage-discharge curves, to examine the effect of tailwater elevation, and to determine the hydraulic capacity of model dams having different cut-off wall heights. It was found that a model dam having an impervious element (cut-off wall) extending only part way to the crest did not have appreciably greater hydraulic stability than a model dam having a cut-off wall extending the full height of the model dam. Use of partial cut-off walls, which do not maximise the available head for a given volume of fill, may not therefore represent the most economical design.

The flume studies indicate that it may be desireable to unify the concepts of 'overflow' and 'flowthrough' in rockfill dams for the hydraulic design of such structures, since significant flowthrough occurs in the downstream shell of 'overflow' rockfill dams.

Additional studies are deemed to be essential in order to formulate design guidelines.

# DESIGN OF LOW-COST FLOWTHROUGH ROCKFILL DAMS AND SPILLWAYS

## 1.0 INTRODUCTION

Since the work of Wilkins (1956, 1963) and Parkin (1963, 1966, 1971) little practical work has been done in the area of flowthrough rockfill dams. In addition, most of the work done in the past on flowthrough dams has been done related view to the construction of such structures in warm climates. However, it is perceived that rockfill structures may be particularly well suited to remote northern environments where concrete spillways are more expensive to build and to maintain.

The flow of water through coarse crushed rock is more rapid than through soil and often does not follow Darcy's law. Non-Darcy flow through a matrix of crushed rock has therefore been the starting point of many previous investigations into the hydraulic behaviour of flowthrough rockfill dams, and it was also the starting point of the experimental program in this study. These more quantitative experiments were followed by hydraulic tests of model rockfill dams in a flume, with a view to qualitatively studying hydraulic performance and stability of flowthrough dams. These investigations were done with a view to qualitatively examining some safety and economic aspects of flowthrough structures.

A number of flowthrough rockfill dams have been constructed in countries such as Australia, Tasmania, and South Africa, and it is hoped that this report represents a contribution toward more fully bringing this technology and its economic benefits to Canada.

## 2.0 OBJECTIVES

Phase I of this study was comprised primarily of:

1. A literature review of the present knowledge of non-Darcy flow through *porous* media, of seepage through rockfill, and of the hydraulic design of flowthrough rockfill dams.

2. A preliminary parametric study of the flowthrough characteristics of rockfill placed in a test cylinder. The influence of such factors as rock size, shape, gradation, degree of compaction, and angularity were to be investigated.

3. Preliminary flume tests to study the hydraulic behaviour of model flowthrough rockfill dams. The results of this component of the study were to serve as a guide for work to be done in Phase II of the research program.

## 3.0 JUSTIFICATION FOR RESEARCH

The program of activities is aimed at studying the flow characteristics of water through rockfill in order to develop procedures for the economical design of low-head embankment dams and spillways. In most small hydro schemes, a dam is required to create adequate head and a spillway is required to safely pass flood flows. While the dam may be comprised of earth, rockfill, or concrete, the spillway is invariably a concrete structure. The cost of concrete structures is often a major component of the total cost of civil works, particularly in remote northern environments. Also, the durability of concrete in the harsh conditions of the northern Canadian environment is adversely affected. It is believed that in many instances the provision of a "flowthrough" rockfill spillway capable of handling the design return period flood event could lead to a significant reduction in the cost of small hydro schemes. As a former Chairman of the Australian Water Research Foundation has noted (1963):

> "The development of the rockfill self spillway dam, in which excess water passes through the dam itself, thereby dispensing with the need for an expensive spillway, is an existing promise of reduced costs of major water conservation projects under conditions favouring this method."

Small-scale hydro schemes are generally located to serve small communities in relatively remote locations. In the Canadian context such works would very likely serve locations which experience severe winter conditions. It is imperative therefore that the engineering design and construction be kept as simple and as maintenance free as possible. Currently most small-scale hydro projects involve small concrete dams, spillways, and diversion structures which have three limitations:

1. Concrete works invariably require considerable site preparation before they can be constructed. A major cost is the excavation to a hard subsoil strata on which the structure can be founded. This also limits the type of terrain on which such structures can be sited.

2. Concrete structures require considerable engineering design and construction skills - not readily available in many northern Canadian communities. Construction skills and equipment are therefore often 'imported' to the area, thereby increasing design and construction costs.

3. The durability of concrete in harsh winter conditions is poor, and periodic maintenance is required. This is especially true of small hydro schemes in remote locations where rigourous control of the quality of the concrete mix and of general workmanship is either lacking or non-existent.

A low-head rockfill dam with a built-in flowthrough section alleviates many of the problems associated with concrete dams. In particular, the low-head rockfill dam requires very little site preparation and can be placed over virtually any soil or rock conditions. The construction work can often be carried out by utilizing the resources available within many northern Canadian communities. Also, sound rockfill is highly resistant to freeze-thaw action. The rockfill dam is deformable and accommodates the imposed stresses and strains. Moreover, in contrast to concrete, rockfill is not prone to cracking.

It is of interest to note that Gordon and Noel (1986) have estimated that the civil works of hydro projects ordinarily account for 60 percent of the cost of a project. They further suggest that, for construction of such civil works in northern Canada, a factor of 1.4 be applied to the cost estimate of the civil site works, making this component responsible for 68 percent of the total cost of the project. It may therefore be expected that the use of flowthrough rockfill dams in northern Canada can reduce the high cost of civil works for small hydro developments in the north.

## 4.0 LITERATURE REVIEW

### 4.1 General Research in non-Darcy Flow.

The more general research area of non-Darcy flow in porous media is important in the hydraulic design of flowthrough rockfill dams for two reasons:

1. In flowthrough dams the quantity of seepage is substantial and intentional, unlike conventional earth dams in which efforts are made to keep the quantity of seepage relatively small. The quantity of flow through a rockfill dam cannot be estimated without the correct non-Darcy flow equation.

2. In flowthrough dams the pore pressure pattern is important because downstream slope stabilization may be required if the point of emergence of the flow is relatively high on the downstream face. This point of emergence cannot be determined without determination of the the parameters in the non-Darcy seepage equation.

Determination of the appropriate non-linear seepage equation is therefore of particular importance in the design of flowthrough rockfill structures. Studies of non-Darcy seepage abound not only in civil engineering literature, but also in chemical, mechanical, and geological engineering publications. A review of the literature, which includes research from these various disciplines, is included for completeness.

The expression for non-Darcy flow is normally given in one of two forms:

$$[1] \quad i = a V^N$$

$$[2] \quad i = a V + b V^2$$

where:

i = hydraulic gradient (dimensionless)
V = the Darcy velocity, also known as the apparent velocity (= Q/A )
a, b = empirical coefficients
N = empirical exponent having a value between 1 (laminar) and 2 (fully turbulent)

Sunada (1964) and Rumer (1969) have noted that [2] can be obtained analytically using analogies from the theory of drag to describe the hydraulic resistance of the porous media. Ward (1964) used the Rayleigh method of dimensional analysis to show that [2] can be arrived at analytically, and further demonstrated that:

$$[3] \quad a = \frac{\mu}{\kappa}$$

$$[4] \quad b = \frac{0.550\,\rho}{k^{1/2}}$$

where:

$\mu$ = absolute viscosity of water
k = the permeability (units of $L^2$)
$\rho$ = mass density of water

The equation resulting from combining [3] and [4] with [2] is then dimensionally homogeneous.

Previous research has been diverse with respect to the type of porous media used. Various authors have developed their own empirical expressions for a given particle type or group of particle types, and these expressions are often at variance with each other (Scheidegger 1956). The hydraulic gradient, i, required to cause flow through a cylindrical packed bed is dependent upon a number of factors (after Rose 1945):

$$[5] \quad i = f(g, \mu, \rho, D, d, n, X, S, G)$$

where:

g = gravitational constant
$\mu$ = absolute viscosity of the fluid
$\rho$ = density of the fluid
D = diameter of the cylinder
d = a diameter representative of the particles
n = porosity of the bed
X = a measure of the surface roughness of the particles
S = particle shape
G = the gradation of the particles

Rose (1945a) investigated high pressure flow through random packings of 1 cm spheres and stated that only a mean resistance law could be defined because each random packing resulted in a somewhat different equation. Further, for a given packing arrangement, Rose (1945a) observed increased scatter in the data as the velocity increased past the laminar flow regime. For the spherical uniform material used, a clear relationship was observed between the hydraulic gradient required to maintain a given velocity, and the porosity of the bed. Parallel curves were presented by Rose (1945b) for materials of increasing angularity. Crawford and Plumb (1986) observed a similar parallel shift

in friction factor-Reynolds number curves as particles increased in angularity.

Brownell and Katz (1949, 1950), seeking as others have done to find a suitable redefinition of the Reynolds number which could universally be applied to porous media, found that porosity was the most important parameter affecting resistance to flow, with particle sphericity and angularity of lesser importance.

It would appear that the equation most commonly accepted in chemical engineering for flow through packed beds is the Ergun (1952) equation. The main virtue of the this equation is that it covers both the laminar and fully turbulent flow regimes (Sissom and Pitts, 1972). The equation can be stated as:

$$[6] \quad \frac{i\,D}{\rho\,V^2} \cdot \frac{n^3}{1-n} = 150\,\frac{1-n}{Re'} + 1.75$$

where:

$\rho$ = density of the fluid
i = hydraulic gradient
V = Darcy velocity
D = particle diameter
n = porosity
Re'= Reynolds number using particle diameter as the characteristic length.

There are three problems with this equation. First, the only parameters used to characterise the porous media are porosity and nominal particle diameter. As will be seen in the results presented in Section 6, porosity and particle diameter are inadequate to individually explain all of the variation in empirical parameters in a governing equation of the form of [1]. Second, use of particle diameter in a Reynolds number is conceptually founded in the analogy to pipe flow (Scheidegger, 1956). Due to the random nature of particle arrangement in ordinary packed beds, it is possible to achieve different results simply by repacking the bed with the same material at the same porosity (Brownell et al, 1950). This second aspect will be discussed further in the context of the discussion of the 'mean hydraulic radius'. Third, it is not acceptable to rearrange equation [6] into the form of equation [2] for estimation of 'i' from 'V' because equation [6] is the result of a regression, and it is not mathematically valid to algebraically rearrange the results of regression analysis (Williams, 1983). If such equations are rearranged an error in estimation will result, with the magnitude of the error increasing severely with decreasing values of the coefficient of determination.

Unfortunately, none of the above mentioned investigators expressed their results in terms of either equations [1] or [2], but rather presented variations of Moody-type diagrams. Although presentation of results in this form is interesting from an academic point of view, this study will not in general use this method of presenting results because it does not lend itself to the application of interest here: that of developing non-Darcy flownets. Further, the Moody-type diagrams appearing in the literature usually contain spurious correlation, giving a misleading visual impression of the degree of correlation between the 'dependent' and 'independent' variables. This is due to the appearance of parameters, such as those characterising the particle bed, appearing both in the modified Fanning friction factor and the modified Reynolds number. Nonetheless, studies using Moody-type diagrams are of interest because they often indicate the relative importance of various parameters pertaining to the porous media, such as angularity.

Another aspect of most previous research using packed columns is the omission of some measure of the wall effect, which is the local increase in velocity near the wall of the column. Dudgeon (1967) has stated that the most important reason for the wall effect phenomenon is the decrease in porosity near the wall, which is due to the absence of interlocking particles coming from the region outside the wall. This effect can be very significant if the diameter of the column is less than 8 to 10 times the nominal particle diameter. For 1.9 cm (3/4") gravel Dudgeon (1967) found that the porosity near the wall in a 14.3 cm column was 52 %, as opposed to 41 % overall. Using specially designed equipment Dudgeon estimated that the wall effect for the above described packed column resulted in overestimation of the Darcy velocity by 7 percent. The wall effect was found to be relatively independent of the flow rate.

As has been indicated previously, a typical approach in the analysis of non-Darcy flow in porous media is to seek to plot some form of friction factor against a modified Reynolds number. The use of a Reynolds number requires the definition of a characteristic length, which for pipe flow is the pipe diameter. For flow in porous media it is not immediately evident what this length should be. The mean hydraulic radius, m, has been defined (Taylor 1948) as:

$$[7] \quad m = \frac{v_v}{S}$$

where:

S = surface area of the particles
(= surface area of the voids)
$v_v$ = volume of the voids

The parameter 'm' is intended by investigators such as Wilkins (1956) to be a measure of the mean pore diameter within the bed. Scheidegger (1956) argues that even if 'm' is representative of the mean pore diameter, the analogy that is being employed is that of pipe flow. Since no rapid transition from laminar to turbulent flow occurs in porous media, as is seen in pipe flow, it is argued that there is no reason to expect the pipe flow analogy to yield consistent results. Nonetheless, many investigators have used either the parameter 'm', or porosity combined with particle diameter in a modified Reynolds number with success (Rose 1945; Brownell and Katz 1947; Wright 1958, Crawford and Plumb 1986). Figure 1 gives an example of a Moody-type diagram typical of packed column studies.

A key conceptual element in dealing with non-Darcy flow in porous media is understanding the mechanisms by which head loss occurs as the velocity of flow increases. Wright (1968) and Sunada (1964) have done much to elucidate these fluid processes. The existence of the 'steady inertial' and 'turbulent transition' regimes (Wright 1968) for flow in porous media do not generally exist in pipe flow. For the 'steady inertial' regime the convective accelerations in the flow are important and cause the relationship between hydraulic gradient and velocity to be non-linear. These convective accelerations, caused by the tortuosity of the paths of flow, actually suppress true turbulence in this regime. At higher velocities the 'turbulent transition' regime begins, in which random micro-velocity fluctuations occur. These micro-velocity fluctuations cannot be measured directly without relatively sophisticated equipment, such as with hot wire anemometry. In this regime the hydraulic gradient is proportional to nearly the square of the velocity.

The very gradual transition to fully turbulent flow, a transition which is probably unique to any given porous media, makes it difficult to produce quantitative scaling laws for inferring the behavior of prototype system from the behavior of a model. For typical pipe flow problems the behavior of a prototype-sized physical system can be physically modeled as long as it is ensured that turbulence is fully developed in the model. If the points representing the model pipe conditions on the Moody diagram are in the 'flat' region where the friction factor is no longer dependent on the Reynolds number, turbulence is fully developed. For flow through coarse porous media the friction factor is Reynolds number dependent for a wider

range of velocities than for pipe flow. This makes it more difficult to produce prototype behavior on a model scale since prototype flow could be expected to be more turbulent ('N' in eqn [1] approaching two). The model however, will tend toward being more laminar ('N' in eqn [1] closer to one). Gerodetti (1981) felt that the flow regime in his model dam was in the transition zone between laminar and turbulent, but felt that the flow regime in the prototype would be fully turbulent. Such differences will affect the validity of conclusions about prototype behaviour as based on scale models of flow through porous media.

Basak (1977) presented a review of the literature on non-Darcy flow equations. He identified five flow regions, two of which are potentially important for studies of flow through coarse rockfill. These are the non-Darcy post-linear laminar zone, and the non-Darcy post-linear turbulent zone. According to Basak, equation [1] can be used to describe the first zone but not the second zone. Judging from the success that previous investigators have experienced using equation [1] it would seem that the flow regime is most often in the non-Darcy post-linear laminar zone.

Basak presented 17 equations, developed by previous researchers, expressed in the following form:

$$[8] \quad V = M\, i^{k} \quad \text{(cm-sec units)}$$

where M, k are empirically determined and V and i are defined as for equations [1] and [2]. Equation [8] is simply equation [1] stated another way.

The diameter and porosity associated with each study was also presented. It is generally accepted that the coefficient 'M' in equation [8] is dependent on the nature of the matrix of particulate matter through which flow is taking place, while the exponent 'k' is dependent on the degree of departure from laminar flow.

Basak's summary of equations exhibited small variance in the value of 'k' but the coefficient 'M' was found to be much more variable. Based mainly on the literature review of Basak (1977), Figure 2 (a) presents a plot of the coefficient 'a' in equation [1] against nominal particle diameter. Figure 2 (b) presents a plot of the coefficient 'a' against porosity. It may be that for a given study with a consistent type of material, some representative diameter of the particles is probably adequate for the determination of 'a'. However, different studies have resulted in very widely differing values of 'a' even when diameter is considered. The conclusion is that diameter and porosity are individually inadequate for the characterisation of the

parameter 'a', and hence other characteristics of the porous media must be quantified.

## 4.2 Empirical Research in Flowthrough Rockfill Dams

As with more conventional spillways, there is a limit to the discharge that can be safely passed by the structure. This is also true of flowthrough rockfill spillways, and the quantity of discharge that can be safely passed is related to the size and grading of the rockfill (Parkin 1963).

There are three areas of concern in the hydraulic design of rockfill dams:

1. The water surface profile within the dam, with particular attention to the points of hydraulic control within the structure, if any.

2. The internal pore pressures, especially near the point of emergence on the downstream face. At this particular section the magnitude of the exit velocity is important because seepage tends to destabilise the rock at this location. The pore pressure pattern within the dam is also important with regard to the stability of the downstream shell against circular slippage or pseudo-rotational failure.

3. The flow velocities within the dam, especially over the crest of the inbuilt spillway, if any. This is directly related to the discharge capacity of the dam.

In order to deal with the above three areas, two basic tools are needed:

1. An energy loss equation of the type given by [1].

2. A means of generating a flow net, such as by sketching, or by a finite difference or finite element method.

These key aspects of the hydraulic design of flowthrough rockfill structures will be discussed in the following sections. In this regard the research of Wilkins and of Parkin is of primary significance. More recent literature summarizing practical design methods for flowthrough rockfill structures, such as those of Thomas (1976) and Leps (1973), are in fact largely drawn from the work of Wilkins and Parkin.

4.2.1 The Research of J.K. Wilkins

Prior to the work of Wilkins' (1956), engineers designing flowthrough rockfill dams apparently did so without any hydraulic design methodology. The unavailability of hydraulic design methods was due to the non-Darcy nature of flow in coarse rockfill. The proposal to construct the Laughing Jack and Wayatina B rockfill dams in Tasmania initiated Wilkin's research under the auspices of the Tasmanian Hydroelectric Authourity. Experiments were performed in packed columns and on a small model dam 0.61 m (2 feet) high in a flume. For most of the experiments crushed dolerite was used as the porous media. Ordinary glass marbles were also used in the smaller column to examine in a limited manner the effect of angularity on head loss. Three equations resulted:

[9] $V = 32.9\ m^{0.5}\ i^{0.54}$ for stone in packed columns.

[10] $V = 46.5\ m^{0.5}\ i^{0.54}$ for marbles in packed columns.

[11] $V = 42.0\ m^{0.5}\ i^{0.54}$ for stone in a flume.

where V: Darcy velocity in the porous media in inches/sec.
i: hydraulic gradient.
m: hydraulic mean radius, defined as (volume voids)/(surface area of particles), in inches.

Wilkin's main contributions can be considered to be as follows:

1. Use of a simplified form of equation [9] to construct 'turbulent' flow nets. This contribution is still in use (eg. Gerodetti 1981).

Figure 3 illustrates, for an isotropic porous media, the difference between an ordinary Darcy flow net and a 'turbulent' flow net. A well-known characteristic of ordinary laminar flownets is that each panel or element in the mesh is square-like, with the mean distance between flowlines (denoted 'b' in [12]) equal to the mean distance between equipotential lines (denoted '$\ell$' in [12]) for a given individual element. This is not the case for non-laminar flow. For Darcy (laminar) flownets it is known that N = 1 whereas for flow beyond the laminar regime N > 1.

[12] $$\ell = b^{N}$$

where:

$\ell$ = mean distance between equipotential lines within a given mesh element
b = mean distance between flow lines within a given mesh element
N = empirical exponent as defined in eqn [1]

Relation [12] means that for a given hydraulic gradient the mean distance between flowlines is less than the mean distance between equipotential lines for any individual element in a non Darcy flow net mesh. Decreasing the mean distance between flowlines relative to the mean distance between equipotential lines implies a greater number of flow paths. The greater number of flow paths is consistent with a greater total quantity of seepage for the semi-turbulent nature of the flow.

As with laminar flow nets, much can be learned quantitatively and qualitatively from the two dimensional flow net of a dam. First, the quantity of seepage can be estimated if the empirically determined parameters in the governing flow equation are known. Second, the pore pressures within the dam can be estimated for any location. This is particularly important when considering the stability of the downstream slope of the structure. Third, an approximation can be made as to the location of the highest point where water emerges from the downstream face. This indicates the region on the downstream face for which anchorage is most important.

2. Another important contribution of Wilkins' was his description of the modes of failure of rockfill dams, also confirmed by Sandie (1962). For relatively steep downstream slopes the primary mode of failure was found to be progressive sloughing of the downstream face. If the downstream face was anchored and a sufficient head was applied, a secondary mode of failure was found to be a massive deep-seated circular slippage or sliding, near the foundation.

3. Based on his qualitative observations on types of failure, Wilkin's suggested means of anchoring large rockfill dams to ensure their survival and, in later research (1963), presented detailed anchor schemes based on more formal slip-circle stability analyses.

Wilkin's research raised two important questions. First, how can the discharge through the dam and the height above tailwater of the highest point of emergence of seepage on the downstream face ($h_e$) be computed directly ? Second, since it appears that the hydraulic mean radius 'm' (Taylor 1948) is not entirely adequate to characterise the porous media, how can the particulate media be properly

characterised so that its hydraulic behavior can be described independent of experimental work.

### 4.2.2 The Research of A.K. Parkin

Parkin (1963) provided an answer to the question regarding the height of the point of emergence of the seepage on the downstream face, but the question of how the geometry of the porous media affects its non-Darcy hydraulic behavior remains a difficult problem. Parkin's research at the University of Melbourne in Australia was in good agreement with Wilkin's results. His experiments were done both with packed columns and with a model dam in a flume 1.2 m wide. The model dams had a sloping internal impervious wall set parallel to the upstream face, but not reaching the crest. Parkin chose to simplify and restate his primary result as:

$$[1] \quad i = a V^{N} \quad \text{(cm-sec units)}$$

For 19 mm (nominal 3/4") aggregate,

where:

a = 0.0116, for V defined as Darcy velocity.
N = 1.86

The exponent in the above equation compares well with that found by many other investigators (see Basak 1977).

Parkin's (1963) main contributions are as follows:

1. The delineation of four zones of flow within rockfill dams having inbuilt spillways of the type described above. These four zones are shown schematically in Figure 4(a). Zone one normally determines the discharge capacity of the dam.

2. The identification of the two key points of hydraulic control in a rockfill dam with an inbuilt spillway. These are the depth over the inbuilt spillway, $h_c$, and the height above tailwater of the highest point of emergence of seepage on the downstream face, $h_e$. Parkin (1963) also showed experimentally that if the height of the inbuilt spillway is low relative to the height of the dam, the height of the point of emergence on the downstream face begins to be affected by the depth of water over the crest of the inbuilt spillway.

3. The presentation of a method of directly calculating the discharge over the inbuilt spillway and therefore through the dam.

4. The incorporation of equation [1], in vectorial form, into the LaPlacian continuity equation which resulted in a partial differential equation for 'turbulent' flow in porous media. This equation would later be successfully used by Curtis and Lawson (1967) to arrive at numerical solutions to describe 'turbulent' flow in porous media.

As has been indicated, for rockfill dams with inbuilt spillways, two quantities are of particular importance in design. These are $h_c$, the height of water over the crest resulting from a given discharge, and $h_e$, the exit depth. The following is a review of Parkin's (1963) method of computing these quantities.

4.2.2.1 The crest head, $h_c$

Parkin (1963) found that the hydraulic gradient over the crest of the inbuilt spillway was 0.82 for 3/4 inch crushed rock. This gradient is then substituted in the governing flow equation:

$$[13] \quad 0.82 = a V^N$$

The Darcy velocity, V, can then be solved for in [13], knowing 'a' and 'N'. From [14] the crest head $h_c$ can then be determined:

$$[14] \quad Q = V h_c W$$

where:

Q = desired flood capacity
W = width of the spillway
$h_c$ = crest head; the height of water above the impervious element.

This method of determining $h_c$ has two limitations:

1. There is no reason to believe that the hydraulic gradient over the crest of the weir is 0.82 for all types of rock.

2. The method depends on the value of 'a', and to a lesser extent, 'N'. The value of 'a' will depend on the material used. Parkin (1966) presented a nomogram to assist in estimating the value of 'a' from porosity and the mean surface area per unit volume of the particles, but suggested that the values of 'a' and 'N' should be determined experimentally when approaching a design problem. In view of the fact that the variability of 'a' is far greater than the variability of 'N' it would appear that it is relatively more important to determine 'a' for design problems.

4.2.2.2 The critical exit depth, $h_e$

The formula presented by Parkin (1963) for determining $h_e$ is as follows:

$$[15] \quad h_e = q\,(a/\sin \beta)^{(1/N)}$$

where:

$h_e$ = critical exit depth
q = width-specific discharge
a, N = as defined for eqn [1]
$\beta$ = slope of downstream face of dam, measured from the horizontal

The development of this equation assumes that the slope of the water surface profile on the downstream face is equal to the slope of the downstream face of the dam. For this case the flow must be uniform and $\sin \beta = \tan \beta$. It is evident that the value of $h_e$ depends on 'a', a quantity highly dependent on the type of rock material, as has been shown.

The work of Parkin (1963) is significant in that it most closely approaches a design manual for flowthrough rockfill dams. Parkin's (1963) work can be summarized as being largely an extension and verification of the work of Wilkin's, again for rock of nearly uniform gradation of a particular angularity and size range. Parkin likened his empirical equation to that of Hazen-William's for flow in pipes. His equation does not give a broad picture of how roughness, the friction factor, and the Reynolds number are related, as does the Moody diagram does for pipe flow, but does give reasonable results for a restricted range of conditions. Parkin's (1963) results were summarized in his 1966 ASCE paper, in which a graph was presented showing the conditions under which a rockfill dam will be stable or unstable. This stability graph is given in Figure 4(b).

Johnson (1971) was fortunate to have access to fairly comprehensive data on the conditions of failure of a partially completed rockfill dam in California, and used this data to evaluate some of the work of Parkin. With independent hydrologic and hydraulic estimates Johnson developed a rating curve for the dam that failed. Based on photographs of the area of water emerging from the seepage face he computed Parkin's 'a' coefficient as 1.75, which is in reasonable agreement with Parkin's value considering that the Johnson did not have detailed knowledge of the nature of the rockfill other than that it was 'end-dumped and homogeneous'. By considering hypothetical heights of the inbuilt spillway, Johnson proposed a modified form of

Parkin's (1966) stability graph and showed that the dam failure which he considered did reside in the unstable zone for the flow conditions at failure.

## 4.3 The Overflow Condition

A choice of emphasis is apparent in the literature regarding the type of hydraulic design for rockfill dams having no special concrete spillways. A number of engineers have opted for a design which emphasises overflow. That is, the dam is designed to pass the probable maximum flood with most if not all of the excess water passing over the crest and downstream face of the dam. Others have designed and built rockfill dams with the intention that the overflow condition should not occur, so that the the probable maximum flood passes through the dam. In any case, as Leps (1973) has noted in commenting on the work of Parkin:

> "...it will seldom be prudent to permit flow through a rockfill without reinforcing or buttressing the downstream face."

In the same way that even a conventional concrete spillway may on rare occasion be subjected to a flood of a greater magnitude than the design flood, so too flowthrough rockfill dams may be subjected to such rigours. It is in this regard that it is prudent to consider the hydraulic behavior of a flowthrough rockfill dam during overflow.

When a rockfill dam is overtopped the water flowing down the downstream slope may cause stones on this slope to dislodge. If a sufficient number of stones dislodge this effect may become self inducing; the stones rolling down the face tend to set other stones in motion which are already in a state of incipient motion under the turbulent flowing water. This is known as "unraveling" failure (Gerodetti 1981). Unraveling failure usually leads to a sloughing-off of material on a larger scale, and the dam is often breached not long afterward (Wilkins 1963).

The usual method of protecting against unraveling failure is to place wire mesh on the downstream face of the dam and anchor this mesh at a number of locations with tie-back bars projecting back into the body of the dam. For this type of design the integrity of the mesh is crucial, and the mesh openings must be smaller than the smallest rock on the downstream face of the dam. The failure of the partially completed Cethana dam on November 15, 1968 in Tasmania was due to damage to the mesh during an overtopping flood. Stones rolling down the face damaged the mesh, finally resulting in the washing away of 20,000 cubic yards of material. Similarly, the failure of Paloona dam in Portugal on February 24, 1969 was due to the wire mesh being

badly torn by a log carried by the flood (HEC Tasmania, undated).

Incidents like these have generated interest in the design of stable rockfill slopes which can be submerged, without appreciable unraveling, by high velocity turbulent flow. Guskov and Pravdivets (1987) and Hartung and Schuerlein (1970) have presented equations for determining the stable rock size required for flow in steep open channels with a view to rockfill dam design. Ulrich (1987) has more recently presented a similar equation in which the angle of repose is replaced by the so-called bearing angle of the rock. Unfortunately the above methods do not consider the destabilising effect of the seepage forces acting on the external rock face. This outward seepage force must be considered if such flow of water can occur (Stephenson, 1979), and has been included in investigations by Sparks (1967) and Gerodetti (1981). This is an important point because even a rockfill dam with an impervious wall extending to the crest has significant seepage through the downstream shell and out of the downstream face, as illustrated in Figure 5 (a). Because of this phenomenon the term 'overflow' is to some degree a misnomer for rockfill dams undergoing seepage through the downstream shell. Figure 5 (b) illustrates the forces acting on a particle on the downstream face of a dam undergoing both overflow and flowthrough.

The research of Olivier (1967) emphasised so-called overflow design, with flowthrough behaviour occurring only in the downstream shell. In order to avoid destabilisation of the downstream slope Olivier proposed very flat downstream slopes of the order of 1V to 5H, particularly at the toe of the dam, so as to help ensure submergence by the tailwater of the point of emergence of the seepage. If some dislodgement of material occurred, Olivier observed that at these gradual slopes the stones simply rearranged themselves into a series of steps which were more stable. The disadvantage of Olivier's approach is that the volume of rock in the dam begins to become very large for such gradual downstream slopes, reducing the economic advantage of the rockfill dam. The steepness of the downstream slope can be safely reduced by using larger rock for this surface (Sparks, 1967). This may be an economical solution if large rock is abundantly available and can be placed cheaply.

The key factors affecting the relative merits of overflow versus flowthrough design for small hydro development are economics and safety. The starting point for such a discussion pertains to two important parameters: the design flood, $Q_{pmf}$, which is essentially imposed, and the amount of head, $H$, desired for the small hydro scheme. The cost of any embankment dam of a given configuration is proportional to the height of the dam. For a dam which is

strictly flowthrough, part of the total height must be used for passing the design flood. This is that fraction of the total height above the impervious element or cut-off wall. The remainder of the height is available for storing water behind the impervious element.

For a dam which emphasises overflow, the entire height of the embankment is available for storing water. It is therefore evident that the relative 'spillway' capacities of these two types of dams, keeping in mind the relative heights of the dams, is of critical importance. This is because part of the cost of the fill for a flowthrough dam is associated with providing for flowthrough capacity above the impervious element. If the impervious element extends to the crest, as in overflow design, the storage is maximized for a given height of dam. However, two factors tend to mitigate against the choice of overflow design:

1. In the absence of a protective mesh of the downstream face, relatively flat slopes are needed to ensure the hydraulic stability of the rock on this face (Olivier 1967). This tends to increase the cost of the dam because its volume becomes great.

2. If steeper slopes are used to reduce the volume of fill, a protective mesh covering the downstream face is essential. Such mesh has a limited life span, and as has been indicated earlier, failure of the mesh can mean failure of the dam.

3. For flowthrough dams only extreme events need pass over the dam. Flowthrough dams normally operate with the flow passing through the dam. This design, having relatively low velocities of flow in the voids as compared to the high velocities associated with the overflow condition, is inherently safer.

## 4.4 Numerical Methods for Flow Net Determination

An important application of the partial differential equation developed by Parkin (1963) was demonstrated by Curtis and Lawson (1967). In their study a numerical solution to Parkin's partial differential equation was used to predict lines of equal pressure within a low rockfill bank having no inbuilt spillway. This rockfill bank of 3/8 inch basalt particles was built in a glass-sided flume which was instrumented with piezometer tapping points at regular intervals on the floor beneath the model embankment. The horizontal profile of water over the bank was measured with a carriage-mounted vernier point gauge. The turbulent flowlines within the structure were observed by injecting flourescein dye at various points on the upstream face. In this way the flow net predicted using the numerical solution

to the partial differential equation for 'turbulent' flow could be compared with an observed flow net in the rockfill.

Unfortunately three factors limit the wider applicability of the results of the study by Curtis and Lawson to the analysis of predominantly flowthrough rockfill dams. First, an unusual geometry for the embankment was used. The model embankment in this case was 25 cm high, 1.8 m long, 1.2 m wide, and had vertical upstream and downstream faces. Second, flowthrough was only 5 to 10 percent of the total flow, making this more of a specialized overflow study. Third, no phreatic surface existed within the model dam used. The pressure of the overflowing water was used as a boundary condition for the numerical solution of the partial differential equation. Nonetheless, this study presents a viable experimental and analytical methodology for the study of flowthrough rockfill dams, and could be duplicated using flow conditions and a dam geometry more suitable to consideration of small hydro schemes.

In something of a departure from his previous work, Parkin (1971) suggested the use of the Forcheimer equation to describe turbulent flow in porous media:

$$[2] \quad i = aV + bV^2$$

Parkin used this equation with an integral form of the equation for 'turbulent' flow in porous media to arrive at solutions using the finite element method. The flow net which resulted from this study confirmed what Wilkins (1956) had previously shown, namely: that the non-Darcy nature of the flow resulted in higher hydraulic gradients at the toe, in a higher phreatic surface, in a larger seepage face, and in higher pore pressures within the structure. Based on these considerations and some slope stability analyses using Bishop's method, Parkin presented a useful rule of thumb for the extent of stabilization needed for the downstream face of a rockfill dam.

Parkin's comments on the merits of finite difference and finite element methods in the study of turbulent flow in porous media are worthy of mention. The finite element method was considered to be computationally faster but more complex. Moreover it was also considered to be more suitable for the study of flow where a curved free surface exists because a series of triangular elements can closely approximate a curved boundary. The finite difference method was considered to be slower but computationally simpler. Parkin preferred equation [1] for finite difference solutions and equation [2] for finite element solutions.

However, based on the similarity of Parkin's results with previous work it would appear that little is to be gained by using the finite element method. This will be

especially true if the finite difference scheme of Curtis and Lawson can be improved so that the phreatic surface is determined automatically. Olsthoorn (1985) has described how this can be done for laminar (Darcy) flow, and it is entirely possible that the iterative finite difference method described by Olsthoorn (1985) for defining the phreatic surface can be extended to non-laminar flow through rockfill.

Volker (1969), also working in Australia, performed a study similar to that of Curtis and Lawson (1967) except that the geometry of the model dam more closely approximated the geometry of a typical prototype-sized rockfill dam, and flowthrough the dam was emphasised as opposed to overflow. In accordance with Parkin's (1971) findings, Volker found that equation [2] performed more satisfactorily than equation [1] in conjunction with the finite element method. Using the finite element technique Volker (1969) was able to accurately estimate the location of the free surface and was also able to estimate the discharge within 10 percent of experimental observations. Volker noted however that the existence of a mathematical 'singularity' at the top of the impervious element reduced the accuracy of computed discharges for this numerical method when a impervious element was present. He also noted that the freefall zone described by Parkin (1963) could not be accurately simulated due to its somewhat erratic behavior. Fortunately, the precise configuration of the freefall zone is of little consequence in design (Parkin 1963).

## 4.5 Winter Performance of Rockfill Dams.

There appears to be little in the literature on how flowthrough rockfill dams behave hydraulically under severe winter conditions. However, certain inferences can be made about research needs based on existing knowledge of river ice processes and of the operation of non-rockfill hydroelectric dams in winter.

Three forms of ice are pertinent to later phases of the investigation. First, ordinary sheet ice will probably cover the upstream reservoir. This may have a small effect on the 'flowthrough' behavior of the dam because the velocity distribution under such conditions is known to be different from that encountered in open water (Uzuner 1975). Second, it is known that ice called 'shorefast' ice is the first ice to appear in natural rivers due to the higher rate of heat loss by conduction from the river banks. This type of ice is particularly prevalent where there are rock outcrops from the river banks. This phenomena raises the question of how the hydraulic behavior of a flowthrough rockfill dam will change if ice begins to form in the voids due to this rapid heat loss.

Third, under certain upstream conditions 'frazil' ice can form in the water and move downstream, attaching firmly to anything it comes in contact with. This floss-like ice of neutral buoyancy has caused difficulties for hydro-electric plants during winter operation because it can very quickly block intake structures by adhering to the trash racks (USACE 1982). Very few laboratories have the capability to generate known mass flow rates of frazil since specific conditions of the atmosphere, water quality, and fluid turbulence must be correctly simulated (Osterkamp 1978). Further, no accepted method exists to measure the mass flux of frazil, although attempts have been made (NWRI 1982).

For the cases of ice growth in the voids due to heat loss by conduction, and of frazil accumulation in the voids, a rockfill dam would overtop because flowthrough operation would become impossible.

## 4.6 Summary

1. Most of the existing non-Darcy flow formulae have been developed for small mono-sized rock. Little research has been done to determine the effect of gradation and other parameters descriptive of the porous media on the hydraulic characteristics of rockfill dams, particulary the mean hydraulic radius.

2. Empirical relations and methods exist which permit estimation of overflow and flowthrough characteristics of rockfill dams. If hydraulic conditions are known the stabilization requirements, if any, can be established. The degree to which existing design tools are applicable in a given setting would probably depend on the degree to which the particle matrix at a site is similar to that for which the original equations were developed.

3. The finite difference and finite element methods are powerful tools for analyzing flow through porous media, but at present they still rely on equations [1] or [2] with their empirically determined values of 'a' and 'N'. This also applies to the more approximate but more readily usable flow net method of discharge determination.

4. In the past, much of the hydraulic related work has dealt with slope stability considerations, particularly when the structure is subjected to overtopping. There is clearly a need for further research on the design and operation of flowthrough rockfill structures and on their performance during the winter.

## 5.0 CHARACTERIZATION OF THE POROUS MEDIA

### 5.1 Introduction

Careful characterisation of the porous media is important because the nature of the media greatly affects the hydraulic properties of the media. Beard and Weyl (1973) investigated the effects of texture on the porosity and hydraulic conductivity of beach sands for the laminar flow regime. They demonstrated that well graded sands had lower porosities and lower hydraulic conductivities than uniformly sized sands. Hydraulic conductivity was found to decrease with a decrease in the median grain size of the sand. Both of these observations are intuitively correct. However, no quantitative information could be in the literature relating gradation and hydraulic behavior for flow beyond the laminar regime.

Wright (1968) has stated that the mean shear stress at the surface of the grains in a porous media is a function of (1) the viscosity, density, and characteristic velocity of the fluid, (2) a characteristic pore or grain length, and, (3) the shape, size distribution, and surface roughness of the grains. The porosity of the media becomes important when the macro-velocity of the flow is substituted for the velocity in the pores, and when some characteristic grain dimension is substituted for the characteristic pore dimension.

In this study three different types of material were used in the hydraulic tests:

1. crushed limestone of different sizes, as supplied by a local quarry,
2. a graded subrounded gravel of glacial origin,
3. round glass marbles.

The hydraulic tests performed in the packed column and in the flume will be described in detail in Section 6. For the following discussions of the geometric characteristics of the porous media used in the column and flume tests, a summary of the various parameters can be found in Tables 1 to 3.

### 5.2 Size and Angularity

Table 1 presents the 95 percent confidence intervals for the nominal particle sizes used each of the column tests. The sizes were determined by measuring with calipers the intermediate axis of 25 to 200 particles for each test. Figure 6 (a) gives the definition of the 'a', 'b', and 'c' axes of a particle, and Figure 6 (b) gives an example Zingg

diagram indicating the variability in shape for the material used in column tests 7. Table 2 gives the proportion of the most common particle for the media used in each test. In Table 3 the computed sphericities, the mean shape factors, and an approximate value of roundness for each material (Krumbein, 1938) are presented. The sphericities were computed as:

$$[16] \quad \text{Sphericity} = \left| \frac{\text{Volume of particle}}{\text{Volume of circumscribing sphere}} \right|^{1/3}$$

This is a measure of the shape of the particle in that it indicates how closely the major axes of the particle approximate a sphere. The shape factor is a measure of the oblateness of the particle:

$$[17] \quad \text{S.F.} = \frac{c}{(a\ b)^{1/2}}$$

Figures 7 and 8 show the materials used for the column tests. It is evident that the angularity of the crushed limestone was quite independent of size. However, it was found that the relative proportions of shapes of stones varied from size to size. As the stone size increased more and more of the particles were found to be disk shaped. This could be due to the mechanics of crushing at the quarry. The effect of these parameters on hydraulic behavior will be discussed in Section 6.4.1.

## 5.3 Mean Hydraulic Radius

The mean hydraulic radius (Taylor 1948) was defined by eqn [7]. This parameter is more rational in hydraulic studies because it is a measure of the mean pore diameter in the packed bed. Since the fluid flows through these pores it would seem more reasonable to use this parameter rather than the nominal particle size for the Reynolds number.

In order to compute this parameter a means of measuring the surface area of the particles was needed. Based on similar work done by Hedlin and Collins (1962) stones were coated with varnish and then coated with fine sand. The difference in weight before and after coating was then a measure of the surface area. Calibration was done by coating cubical stones cut specially for this purpose, and results were found to be reproducible within about 10 percent of true areas. The surface area of the material in the column was estimated on the basis of average surface area per unit mass for 25 to 200 particles. Having the surface area and knowing the porosity permitted calculation of the mean hydraulic radius according to equation [7].

Table 3 gives the mean hydraulic radius for each column test.

## 5.4 Gradation

It is important to investigate the effect of gradation on the governing equation for two reasons (Leps 1973):

1. the gradation of stone produced for use in an embankment cannot be cheaply controlled in the field.
2. gradation may occur due to segregation during placement of the rock.

The gradations for the materials used in the hydraulic tests (to be described in Section 6) were determined by grid-by-number analysis in which 25 to 200 particles were measured individually as to their three major axes. This method, based on selecting particles randomly, has been shown by Kellerhals and Bray (1972) to be equivalent, on average, to bulk sieve analysis. Table 2 presents the coefficient of uniformity, $C_u$, for the material used in each column test. The parameter $C_u$ is defined as that size of material for which 60 percent of the total mass of material is larger divided by that fraction of material for which only 10 percent is larger. The larger the coefficient of uniformity the more graded (less uniform) was the material. The grain size curves for the various material used are given in Appendix II. It will be noted that even materials retained on a given screen size had gradations, based on measurements of individual particles. This is due first to the fact that stones do not tend to lie such that they pass through the screen holes, and second to the limited amount of time for agitation of the screens which was necessarily used in each case. The effect of gradation on hydraulic behavior will be discussed in Section 6.4.1.

# 6.0 EXPERIMENTAL INVESTIGATIONS, RESULTS, AND INTERPRETATION

## 6.1 Introduction

Based on the review of the literature it was evident that four parameters pertaining to the structure and geometry of the porous media influence the head loss through that media:

1. The characteristic pore diameter, if one can be said to exist. Porosity and surface area per unit mass of the porous media are important in this regard.

2. The shape of the particles; whether spherical, disk shaped, etc.

3. The gradation of the particles; whether uniformly sized or well graded.

4. The angularity of the particles; whether smooth surfaced or having jagged edges.

A major goal of this study was to examine how the above characteristics of the porous media influence the expression for non-Darcy flow through the media. Relationships were desired between the empirically determined values of 'a' and 'N' in equations [1] and the nature of the porous media through which flow takes place. In this way if the material intended for a rockfill dam can be characterised sufficiently well the governing equation for flow through the dam will be known.

## 6.2 Apparatus

For this phase of the study a specially designed acrylic column was constructed. This column is shown in Figure 9. It is 1 metre long, has an inside diameter of 28.5 cm, and has a volume of 67 litres. The hydrostatic head available from the constant head tank was 2.9 m at the base of the column. This head proved to be sufficient to provide for non-Darcy flow for all the materials tested, but was not sufficient to induce a quick condition in the material in the column. The acrylic column was placed vertically rather than horizontally in order to make the piezotaps readings less sensitive to their exact location within a given diameter of the column. This arrangement was also a more efficient use of space, and made special containment of the porous media at the top of the column unnecessary.

Flow occurred upwards through the rock in the column and passed unrestricted into a weighing tank. Flow was controlled by a valve near the base of the column.

Piezometric pressure was obtained using brass tubes at eight equally spaced locations in the centre of the column, and the pressures readings were precisely measured with a cathetometer at each flow setting. The tips of these piezotaps pointed upward to avoid the influence of the kinetic head and to facilitate determining the exact location of each tap in the column after each test. Maximum Darcy velocities using this apparatus were of the order of nearly 10 cm per second.

A 38 cm wide glass-sided flume was also used in a limited fashion to perform preliminary experiments on model rockfill dams. This is shown in Figure 10 together with the configuration used for the model dam. The tailwater level downstream of the dam was controlled by an aluminum gate at the downstream limit of the flume. Upstream and downstream water levels were measured with a carriage mounted vernier point gauge, and discharge was measured with a 90 degree v-notch weir at floor level.

## 6.3 Methodology

The first step in preparing for a column test was the preparation of the rock material by sieving it to achieve the desired size fraction, and washing it to remove excess fines. This rock was then placed in small quantities (lifts) in the column. After each lift the material was compacted if any compaction was required for the test. Compaction consisted of dropping a tamper of known weight a known height a predetermined number of times. The piezotaps were inserted at intervals of 10.1 cm during the process of adding lifts. There were eight piezotaps in total, as per Figure 9. Once the column was full the piezotubes were connected to piezobank and the pumping of water could begin.

After filling the constant head reservoir the column control valve was opened and the piezotubes were primed by suction. The control valve was then opened to its highest flow setting. This showed whether the piezobank was high enough and helped stabilize loose particles in the matrix. It also indicates whether main water supply rate was sufficient.

At this point the measurements began. The flow was reduced to a low flow setting. The eight piezolevels were read and recorded with a cathetometer, an instrument for measuring heights which has its own Vernier scale. For a given flow setting the time required for 100 kg (or more) of water was measured three times with a stop-watch accurate to 0.2 seconds. The water temperature was recorded to permit accurate determination of the density of the water. In this manner a Darcy velocity was determined by computing the

volume of water which passed through the column of known diameter in a known time.

After each set of eight readings, the flow was increased so that maximum piezometric height increased by about 4 cm. The system was left for a short time to allow it to reach a new equilibrium state. The spirit level on the cathetometer was then releveled if required, and the next readings were recorded. This procedure was repeated until the maximum possible flow was reached. About 12 sets of 8 piezometric pressure readings each were usually taken.

At this point the control valves were shut off and the main pump was shut down. The material in the column was then removed by hand and was spread out on a large board to air dry for a few days.

## 6.4 Results

### 6.4.1 Column Tests

Twelve column tests were performed. Table 4 lists the type of material used in each test. In general the results in the form of equation [1] are comparable to those reported by Parkin (1963) and Wilkins (1956), particularly with regard to the exponent 'N'. Very little scatter was observed in the data, with $r^2$ for the column tests never being less than 0.99.

Tests 1 through 4 were performed to investigate the reproducibility of the flow equation and the effect of compaction on a material having some gradation. Test 5 was performed to study a material having an intermediate angularity. This material was also relatively well-graded and had a low uncompacted porosity. Tests 6 through 8 were performed to study the influence of particle size. Test 9 was performed to examine the effect of gradation. Test 10 on the glass marbles represented a material with the lowest possible angularity, uniformity of particle shape, and a uniform gradation. Tests 11 and 12 were performed to investigate the effect of compaction.

Figures 11 and 12 give the 12 curves for the empirically derived equations, of the form of equation [1]. Since the correlations between 'i' and 'V' are very good all observed data points fall essentially directly on these curves, which were fitted by ordinary least squares regression of the log transformed data. Correction for the bias introduced by the log transformation of the basic data is not necessary because the correlations in this case are very high (Koch and Smillie, 1986). It was found that equation [1] and equation [2] fit the observed data

approximately equally well over the range of observations made in each test, with:

[1] $i = a V^N$, $r^2 = 0.9994$ (mean)

[2] $i = w + a V + b V^2$, $r^2 = 0.9996$ (mean)
w always close to zero.

There was therefore no clear reason for choosing one functional form over the other, based on the data from these experiments.

Table 5 presents the parameters 'a' and 'N' (in equation [1]) for each of the column tests performed. It is evident from the information in Table 5 that, as with the results presented by Basak (1977) (Figure 2), the value of 'a' is more variable than the value of 'N'. The equation resulting from column test 7 done on 19 cm (3/4" nominal diameter) uncompacted crushed limestone, as was used by Parkin (1963), compares well with the result of Parkin (1963):

[Column Test 7] $i = 0.0098 V^{1.86}$

[Parkin (1963)] $i = 0.0116 V^{1.86}$

The difference in 'a' may be due to different wall effects between the two studies. This aspect will be investigated in Phase II of this study.

It is evident from Figure 11 that the curves for tests 1, 2, and 4 are very close together. This indicates that for crushed rock placed with no compaction the resulting equations are similar, and that test results are quite reproducible. This is less variability than was expected, based on the comments of Rose (1945b).

Figure 13 is a plot of the value of the coefficient 'a' for each equation against the mean hydraulic radius for each column test. It was found that plotting 'a' against mean hydraulic radius rather than porosity or particle diameter resulted in a relationship showing a clear trend. In Figure 13 a linear trend trend is evident based on the data collected to date. This can be expressed as:

[18] $a = -0.115 m + 0.037$ $r^2 = 0.95$

where:

a = Parkin's's coefficient in eqn [1]
m = mean hydraulic radius.

Unfortunately, due to problems with the determination of porosity early in this study, the points corresponding to

column tests CT 1 to 4 are not well supported by the data and have yet to be corroborated. Further data will be collected for this material and results will be reported in Phase II of this study.

However, based on the other tests it is clear that compacting a given material causes the value of 'a' to increase linearly with mean hydraulic radius. This type of trend is evident from the results of tests CT6, CT11, and CT12, in which the degree of compaction is being increased on a mono-sized material.

It is also evident that the value of 'a' is seen to decrease as the size of the particle is increased, in the trend CT6 to CT7 to CT8. This is because the same applied gradient results in higher average velocities as the pore spaces become larger. For CT8, the largest material used, the piezometric pressures were observed to fluctuate markedly at higher flow settings. This may indicate the onset of the 'turbulent transition' regime described by Wright (1968).

Gradation has an effect on the value of 'a' in that it too changes the mean hydraulic radius. Based on the results of tests CT 6 to CT 9 it is apparent that grading the material reduces the mean hydraulic radius and therefore the value of 'a'. Moving from CT10 to CT5 in Figure 13 represents a large shift in the gradation of the material, from uniformly-graded to well-graded, keeping the median particle diameter approximately the same. Unfortunately, it also represents an increase in angularity and the introduction of a variety of particle shapes into the media. The effects of angularity and shape alone have therefore not as yet been identified. The points representing CT10 and CT5 do not lie on the line given by equation [18]. This was because the porosity of these materials was naturally much lower than other materials, even with no compaction. This low porosity was due to the shape of these particles being much more round than the crushed limestone, and in the case of CT5, being graded as well. Such points indicate possible physical extremes. Clearly, the information derived from the column tests on crushed limestone is of greater practical importance in the design of flowthrough rockfill dams.

The most encouraging result from these tests has been a fairly clear linear relationship between the coefficient 'a' and the mean hydraulic radius for a given type of material, in this case, crushed limestone. This is important for two reasons:

1. Crushed stone of an angularity similar to that used in this study most closely approximates the kind of material from which rockfill dams are constructed.

2. The mean hydraulic radius can be estimated without hydraulic testing. This means that the governing flow equation of the type given by [1] can be approximated for design purposes for flowthrough rockfill dams constructed out of crushed stone.

Further tests should concentrate on substantiating existing trends and should seek to isolate the effect of particle shape, if possible.

### 6.4.2 Flume Tests

The glass-walled flume used in this study was 38 cm wide and 16 m long. Preliminary flume tests were performed for three purposes:

1. to check the reproducibility of stage-discharge curves for model flowthrough rockfill dams.
2. to use dye to determine flow lines which indicate an actual flow net.
3. to determine the effect of the imposed tailwater elevation on the rating curve.
4. to examine the relative capacities of a model rockfill dam in overflow and flowthrough modes of operation.

Table 4 lists the type of material used in each flume test. Tests 1 and 2 were performed to check the reproducibility of stage-discharge curves for model flowthrough rockfill dams. Figure 14 (a) and (b) show two such rating curves for '1" clear' (nominal diameter) crushed limestone placed in the configuration given in Figure 10. This material is the same as that used for column tests 1 to 4. The material for the model dam in flume test 1 was completely removed and replaced for flume test 2. The equations for these stage-discharge curves are:

$$\text{[Glass Flume Test 1]} \quad Q = 0.0121\ H^{1.71} \qquad r^2 = 0.99$$

$$\text{[Glass Flume Test 2]} \quad Q = 0.0123\ H^{1.71} \qquad r^2 = 0.99$$

These equations are very similar and demonstrate that rating curves do not vary greatly according to the randomness of the packing of the rock, all others factors being equal.

Dye experiments were not successful in that rapid dispersion of the dye occurred within a few centimeters of the point of entry, making it impossible to delineate a flowline. This effect was obviously due to the turbulent nature of the flow within the model dam.

Flume test 2 was also performed to determine the effect of the imposed tailwater elevation on the rating curve.

Figure 15 shows the effect of an imposed tailwater elevation on the stage-discharge curve. It is apparent that as the tailwater elevation is increased the stage-discharge curve becomes flatter and apparently more linear. Also, it was qualitatively observed that an imposed tailwater helped to reduce the amount of material lost from the downstream slope, especially at lower discharges. This effect was also noted by MacMillan (1977) for model dams made of round stone.

The most important aspect of the flume tests was the examination of the relative capacities of model rockfill dams in overflow and flowthrough modes of operation. The model dam for flume test 3 had an impervious element extending the entire height of the structure, so that it functioned as a so-called overflow dam. Figure 16 (a) shows behavior of this model dam for this flume test. The model dam for flume test 4 had a impervious element equal to 53 percent of its height, so that it functioned primarily as an flowthrough dam. Figure 16 (b) shows behavior of this model dam for this test. The material used in both tests was 3/4" (nominal diameter) sieved crushed limestone, probably very similar to those porous media used by various researchers in Australia, such as Sandie (1962) and Parkin (1963).

The maximum capacity of the model dam for flume test 3 was about 8.2 L/s. Beyond this discharge the downstream face of the dam began to seek to re-adjust itself to a more gradual slope, a phenomena noted by Olivier (1967). Material washed beyond the toe of the dam was removed by hand, however, so that a new stable, more gradual, downstream slope was prevented from forming. In this way a complete failure could be observed. At about 10 L/s the shell of the dam downstream of the impervious element failed completely, and relatively large quantities of rock failed by circular slippage and by sliding, and were rapidly washed downstream. This failure mode was also noted by Sandie (1961) for flowthrough dams. As observed by Gerodetti (1981) for overflow dams of this configuration, seepage out of the downstream face occurs as it does for the flowthrough case, contributing to the failure of the dam (see also Figure 5 (a)). The nonlinear seepage is therefore still important for this case.

Preliminary tests indicate that for flowthrough operation failure occurred in the same manner and at about the same discharge as for the so-called overflow case. It is also notable that as long as the flow was contained inside the dam, the dam was relatively stable. The maximum flowthrough discharge for which the flowthrough dam was stable was about 7 L/s. The overflow dam was also stable at this discharge. This test also confirmed that the exclusively flowthrough behavior is linear, as noted by Parkin (1963) and as seen in Figure 16 (b). Further tests

will be carried out in Phase II of this study to confirm the above findings and to investigate the role of rock size on hydraulic stability.

Flume tests 3 and 4 indicate that a so-called flowthrough design with an impervious element extending only partway to the crest of the dam may not necessarily represent a more hydraulically stable design than a design employing a impervious element extending to the crest of the dam. Having a impervious element extending to the crest also maximizes the available head for a given volume of fill. However, as previously noted, the overflow-type design described here and by Olivier (1967) in fact results in a kind of flowthrough behavior within the shell of the dam on the downstream side of the impervious element. As Gerodetti (1981) has noted, this seepage out of the downstream face plays a significant role in the hydraulic stability of particles on the downstream face. This phenomenon therefore represents an important consideration in design since it cannot be assumed that the impervious element extending to the crest nullifies the destabilising effect of rapid seepage forces acting on the downstream external face of the dam. This aspect of hydraulic behaviour requires a detailed investigation in Phase II of these studies.

## 7.0 SUMMARY AND CONCLUSIONS

Phase I of this study has been useful in meeting the objectives stated in Section 2. The following conclusions have important implications for the practical design of low head flowthrough rockfill hydro dams and will serve to help optimize studies planned in Phase II of this work.

1. A review of the literature indicated that empirical relations and numerical methods exist which permit estimation of overflow and flowthrough characteristics of rockfill dams. The results of applying existing design methods are quite dependent, however, on the expression for the equation of non-Darcy flow through porous media. The applicability of a given form of this equation at a particular site will depend on the degree to which the rockfill aggregate at a site is the same as that for which the original equations were developed.

2. It was found experimentally that the equation governing non-Darcy flow was dependent on the properties of the aggregate, particularly the mean hydraulic radius. A preliminary relationship showed that Parkin's coefficient 'a' was much less variable than values reported in the literature, and that this coefficient is strongly related to the mean hydraulic radius.

3. A relationship between Parkin's coefficient 'a' and the mean hydraulic radius is useful in design when dealing with rapid seepage in rockfill since the governing equation of the type given by [1] can be approximated from readily determined quantity and mass-specific surface area information about the porous media used for the fill, rather than by extensive hydraulic experimentation.

4. The preliminary studies undertaken indicate that it may be possible to develop an empirical relationship betweeen Parkin's coefficient 'a' and the mean hydraulic radius which would alleviate the need for extensive and expensive hydraulic testing. It is expected that a relationship such as that given by [18] will be very useful in the design of flowthrough rockfill dams since crushed rock particles tend on average to have similar angularity and mass-specific surface areas.

5. Preliminary experiments in the flume indicated that there may be little advantage in using an impervious element (cut-off wall) extending only part way to the crest, from the point of view of hydraulic stability. This result is important because use of a partial cut-off wall means a loss in available head for the same volume of fill. This result also indicates that it may be desireable to unify the concepts of 'overflow' and 'flowthrough' in rockfill dams for the hydraulic design of such structures, since 'overflow' rockfill dams of the type tested exhibit considerable flowthrough behaviour. Further investigations are necessary to determine the optimum placement of full height cut-off walls and to determine how flowthrough seepage within the downstream shell affects its stability and the stage-discharge curve of the structure.

## 8.0 RECOMMENDATIONS

The following are the recommendations arising from this study. Recommendations 1 to 3 pertain to the flow through packed column studies.

1. The relationship between Parkin's coefficient 'a' and the mean hydraulic radius is at present based on relatively few data points and further tests are recommended in order to confirm this useful relationship.

2. It is recommended that the effect of gradation on the mean hydraulic radius be further investigated in order to develop a quantitative relationship in which:

$$[19] \quad a = f(m, C_u)$$

where:

a = coefficient in $i = a V^N$
m = mean hydraulic radius
$C_u$ = coefficient of uniformity, a measure of the gradation of the material.

3. The effect of the higher seepage velocity near the wall of the test cylinder apparatus should be investigated in order to determine whether this effect introduces significant systematic error in the results.

The following three recommendations pertain to further studies in the glass-walled flume at the University of Ottawa:

4. Further tests are needed on model dams in which the cut-off wall extends to the crest. These studies will emphasise hydraulic stability and determination of stage-discharge curves.

5. Studies should be undertaken to determine the placement of the cut-off wall which maximizes the hydraulic stability of the downstream shell of the dam.

6. Detailed measurements are needed of the pore pressures within the model dams. This will make it possible to test numerical methods of simulating non-Darcy flow nets.

The following recommendations pertain to studies at near-prototype and prototype scale:

7. Tests should be performed on model dams in the large flume in the University of Ottawa hydraulics laboratory. Data derived from such testing will help in assessing the

magnitude of the scale effect in these studies, and to indicate how the scale may alter the level of turbulence in the seepage.

8. There appears to be very little published literature dealing with the subject of the influence of ice on the hydraulic behaviour of flowthrough rockfill dams. It is recommended that studies be undertaken in a large flume at the National Research Council in order to simulate flow under ice conditions. Such a study is essential for the use of the flowthrough concept in the northern Canadian environment.

9. It is finally recommended that measurements should be undertaken in the field of full-scale flowthrough rockfill structures in Canada. Such field data would be invaluable in validating or modifying the design techniques developed in earlier components of this study.

## 9.0 REFERENCES.

Basak, P., 1977. Non-Darcy flow and its implications to seepage problems. Journal of the Irrigation and Drainage Division, vol.103, IR4, Dec., p. 459-473.

Beard, D.C. and Weyl, P.K. 1973. The influence of texture on porosity and permeability of unconsolidated sand. Amer. Assoc. Petrol. Geol. Bull., p. 349-369.

Brownell, L.E., and Katz, D.L., 1947. Flow of fluids through porous media. AICE Chemical Engineering Progress, vol.43, Oct., p.537-548.

Brownell, L.E., Dombrowski, H.S., and Dickey, C.A., 1950. Pressure drop through porous media. AICE Chemical Engineering Progress, vol.46, Aug., p. 415-422.

Crawford, C.W., and Plumb, O.A., 1986. The influence of surface roughness on resistance to flow through packed beds. ASME Journal of Fluids Engineering, vol.108, Sept., p.343-347.

Curtis, R.P. and Lawson, J.D. 1967. Flow over and through rockfill banks. Journal of the Hydraulics Division, Sept., vol. 93, HY5, p. 1-21.

Dudgeon, C.R. 1966. Wall effects in permeameters. ASCE Journal of the Hydraulics Division, vol.93, HY5, Sept., p.137-148.

Ergun, S., 1952. Fluid flow through packed columns. Chemical Engineering Progress, vol.48 no.2, p.89.

Gerodetti, M. 1981. Model studies of an overtopped rockfill dam. Water Power and Dam Construction, Sept., p. 25-31.

Gordon, J.L. and Noel, R.C.R., 1986. The economic limits of small and low-head hydro. Water Power and Dam Construction, April, p. 23-26.

Guskov, G.D., and Pravdivets, Y.P., 1987. Model of stability of revetments on earth spillway slopes. Hydrotechnical Construction (translated from Russian), vol.20, no.7, Jan., p.405-411.

Johnson, H.A., 1971. Flow Through Rockfill Dam. Journal of Soil Mechanics & Foundations Div., vol.97, SM2, Feb., pp.329-340. Discussion by Kluber and Breth, SM11, Nov. 71, p. 1609-1612.

Kellerhals, R., and Bray, D.I., 1971. Sampling procedures for coarse fluvial sediments. ASCE Journal of the Hydraulics DIvision, vol.97, HY8, Aug., p.1165-1180.

Koch, R.W., and Smillie, G.M. 1986. Bias in hydrologic prediction using log-transformed regression models. AWRA Water Resources Bulletin, vol.22, no.5, p.717-723.

Hartung, F., and Scheuerlein, H., 1970. Design of overflow rockfill dams. Vol. 1 of Proceedings of Tenth International Congress on Large Dams, Montreal, Quebec, June 1-5, p. 587-595.

Hedlin, C.P., and Collins, S.H. 1962. A method of measuring the surface area of granular material. Can. Jour. Chem. Eng., Feb, p.49-51.

Hydro Electric Commission, Tasmania, undated. Failure of Cethana Dam, flood breach of a partly completed rockfill dam. HEC, p.23-36.

Leps, T.M. 1973. Flow through rockfill, in Embankment-Dam Engineering, R.C. Hirschfeld, Ed., John Wiley and Sons, p.87-107.

MacMillan, G.R., 1977. Stability of a rockfill dyke with flowthrough and overflow. Senior report, Dept of Civil Eng., Univ. New Brunswick, Fredericton, N.B., 76 pp.

National Water Research Institute, J.S. Ford, Ed., 1982. Prototype Designs for Surveys and Measurement in an Aquatic Environment. NWRI Contribution 84-25 ES550, National Water Research Institute, Burlington, Ont., 41 pp.

Olivier, H. 1967. Through and overflow rockfill dams - new design techniques. Proc. Inst. Civ. Eng., paper no. 7012, vol.36, p. 433-471.

Olsthoorn, T.N., 1985. Computer notes - the power of the electronic worksheet: modeling without special programs. Groundwater, May-June, p. 381-390.

Osterkamp, T.E., 1978. Frazil formation: a review. ASCE Journal of the Hydraulics Division, vol.104, HY9, Sept., p. 1239-1255.

Parkin, A.K., 1963. Rockfill dams with inbuilt spillways. Part I, Hydraulic Characteristics. Bull. 6, University of Melbourne and Water Res. Found. of Australia. March, 88 pp.

Parkin, A.K., Trollope, D.H., and Lawson, J.D. 1966. Rockfill structures subject to water flow. Journal of the Soil Mechanics & Foundations Division, vol. 92, SM6, Nov., p. 135-151.

Parkin, A.K., 1971. Field solutions for turbulent seepage flow. ASCE Journal of Soil Mechanics and Foundations, vol.97 SM1, Jan., p.209-218.

Rose H.E., 1945a. An investigation into the laws of flow of fluids through beds of granular materials. Proceedings of the Institute of Mechanical Engineers, War Emergency Issues 1-12, p.141-147.

Rose H.E., 1945b. The isothermal flow of gases through beds of granular materials. Proceedings of the Institute of Mechanical Engineers, War Emergency Issues 1-12, p.148-153.

Rose H.E., 1945c. On the resistance coefficient-Reynolds number relationship for fluid flow through a bed of granular material. Proceedings of the Institute of Mechanical Engineers, War Emergency Issues 1-12, p.154-168.

Rose H.E., 1948. Further Researches in fluid flow through beds og granular materials. Proceedings of the Institute of Mechanical Engineers, p.493-511.

Rumer R. R. Jr., 1969. Resistance to flow through porous media, Chap.3 in Flow through Porous Media, R.J.M. De Wiest, Ed., Academic Press, New York, p.91-108.

Sandie, R.B., 1962. A laboratory investigation of self spillway rockfill dams. Thesis, M.Eng.Sc., Univ. of Melbourne.

Sissom, L.E., and Pitts, D.R., 1972. Elements of transport phenomena. McGraw-Hill Co., New York, 814 pp.

Stephenson, D., 1979. Rockfill in Hydraulic Engineering. Elsevier Scientific, Amsterdam, 215 pp.

Sunada, D.K., 1965. Laminar and turbulent flow through porous media. PhD thesis, University of California at Berkeley, 74 pp.

Taylor, D.M., 1948. Fundamentals of Soil Mechanics. John Wiley & Sons, New York.

Scheidegger, A.E., 1974. The physics of flow through porous media. MacMillan, New York.

Ulrich, T., 1987. Stability of rock protection on slopes. ASCE Journal of Hydraulics, vol.113, no.7, July, p.879-891.

United States Army Corps of Engineers Hydraulic Engineering Centre, 1982. Ice engineering, an engineering manual. Publ. EM 110-2-1612.

Uzuner, M.S., 1975. The composite roughness of ice covered streams. Journal of Hydraulic Research, vol.13, p.79.

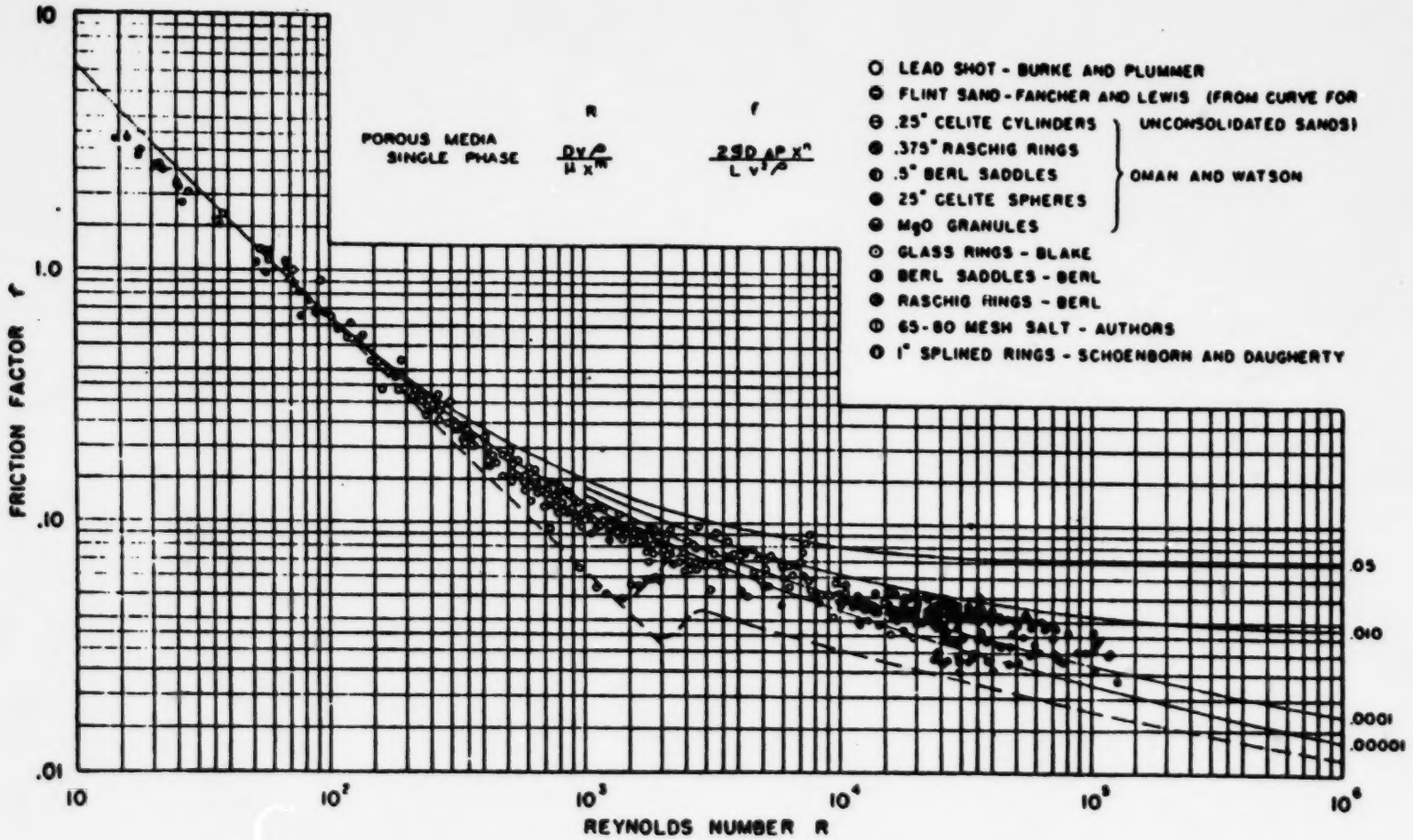
Volker, R.E., 1969. Nonlinear flow in porous media by finite elements. ASCE Journal of the Hydraulics Division, vol.95, HY6, Nov., p.2093-2114.

Weiss, A., 1951. Construction technique for passing floods over earth dams. Transactions, ASCE, vol.116, p.1158.

Wilkins, J.K., 1956. The flow of water through rockfill and its application to the design of dams. Proc. of 2nd Australia-New Zealand Conference on Soil Mechanics and Foundation Engineering.

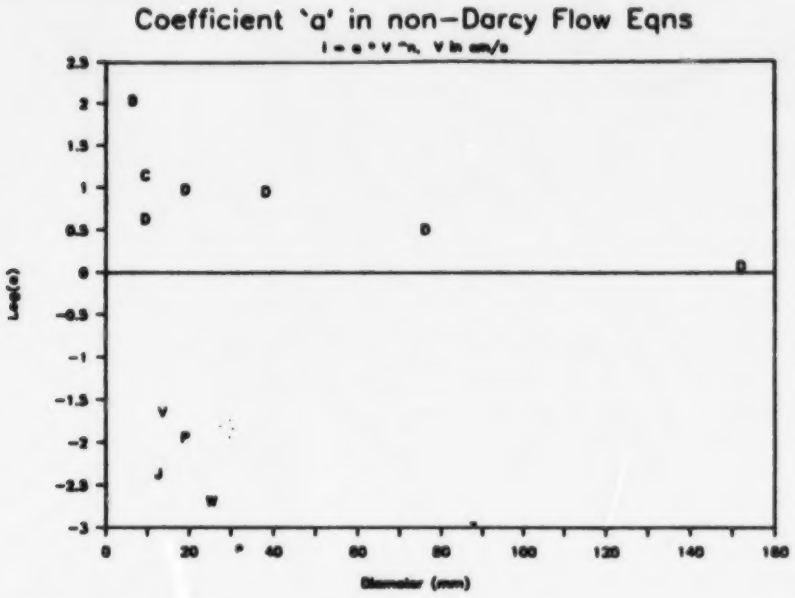
Wilkins, J.K., 1963. The stability of overtopped rockfill dams. Proc. of 4nd Australia-New Zealand Conference on Soil Mechanics and Foundation Engineering, Aug. 19-23, p.1-7.

Wright, D.E., 1968. Non-linear flow through granular materials. Journal of the Hydraulics Division, vol.94, HY4, paper 6018, July, p.851-872.

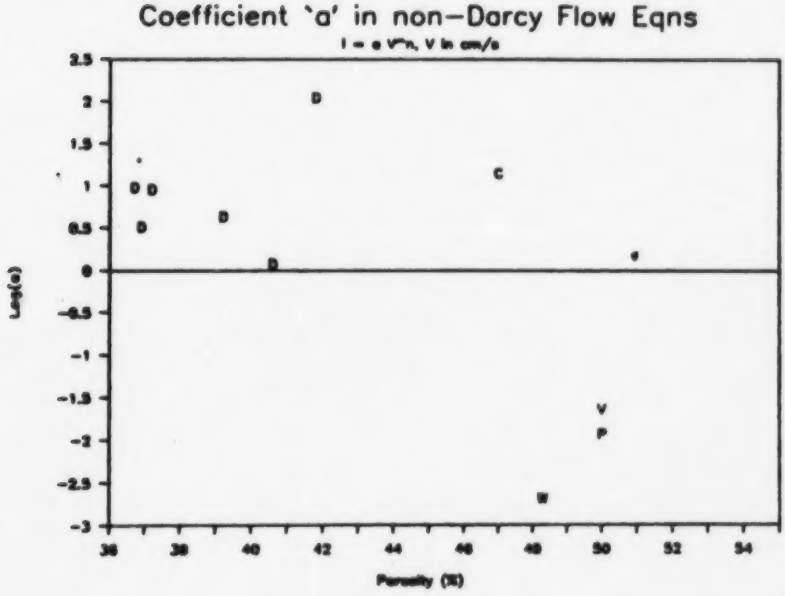



Source: Brownell, L.E., and Katz D.L., 1947. Flow of fluids through porous media. AICE Chemical Engineering Progress, vol.43, Oct., p.537-548.

Figure 1 Example Moody-type diagram for flow through packed column.



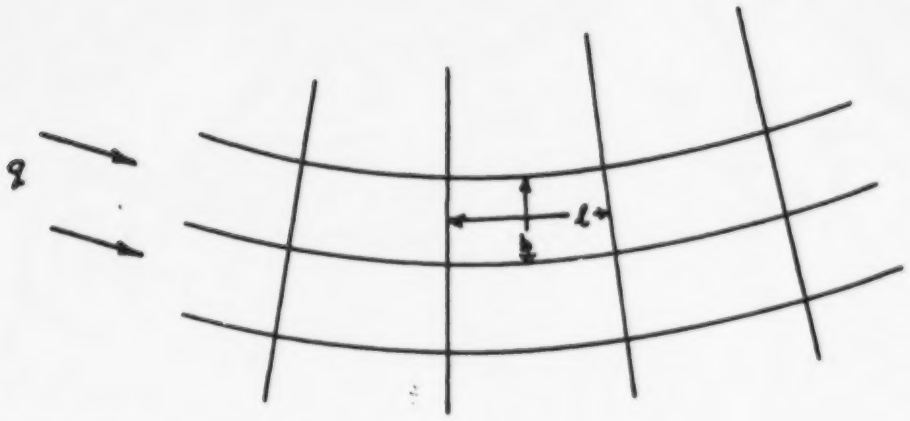

(a) Log(a) vs particle diameter.



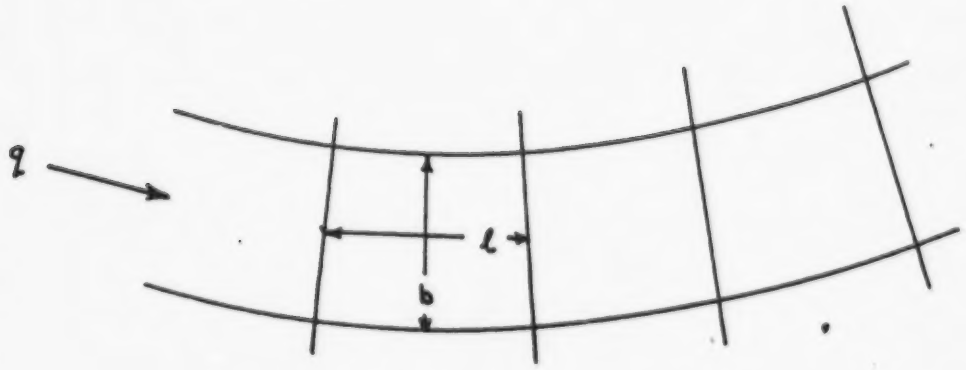

(b) Log(a) vs porosity.

Primary source: Basak, P., 1977. Non-Darcy flow and its implications to seepage problems. ASCE Journal of the Irrigation and Drainage Division, vol.103, IR4, Dec., p.459-473.

**Figure 2** Log(a) vs particle diameter, porosity.

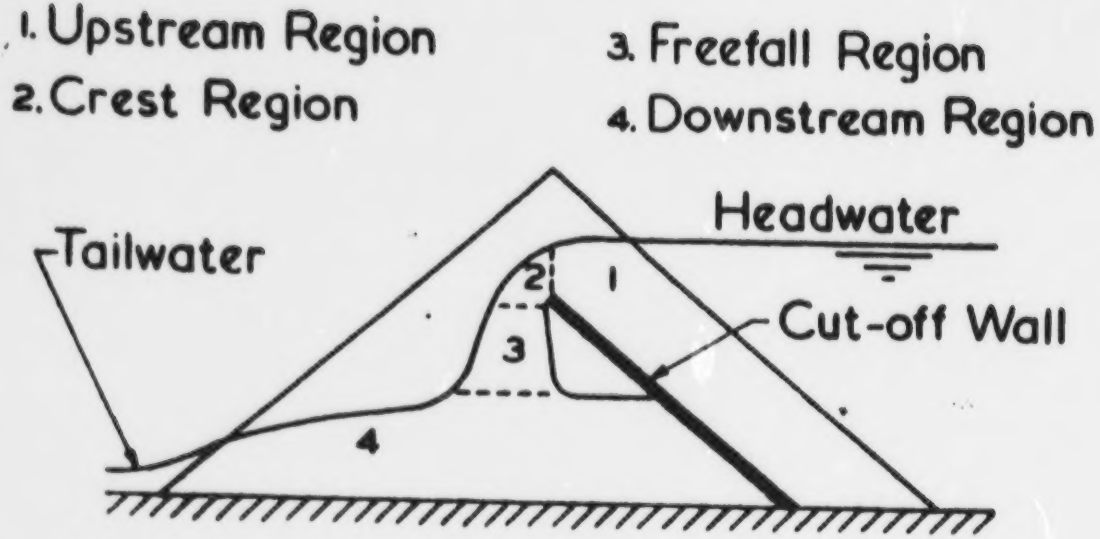



(a) Mesh elements in a non-Darcy flow net.




(b) Mesh elements in a Darcy flow net.

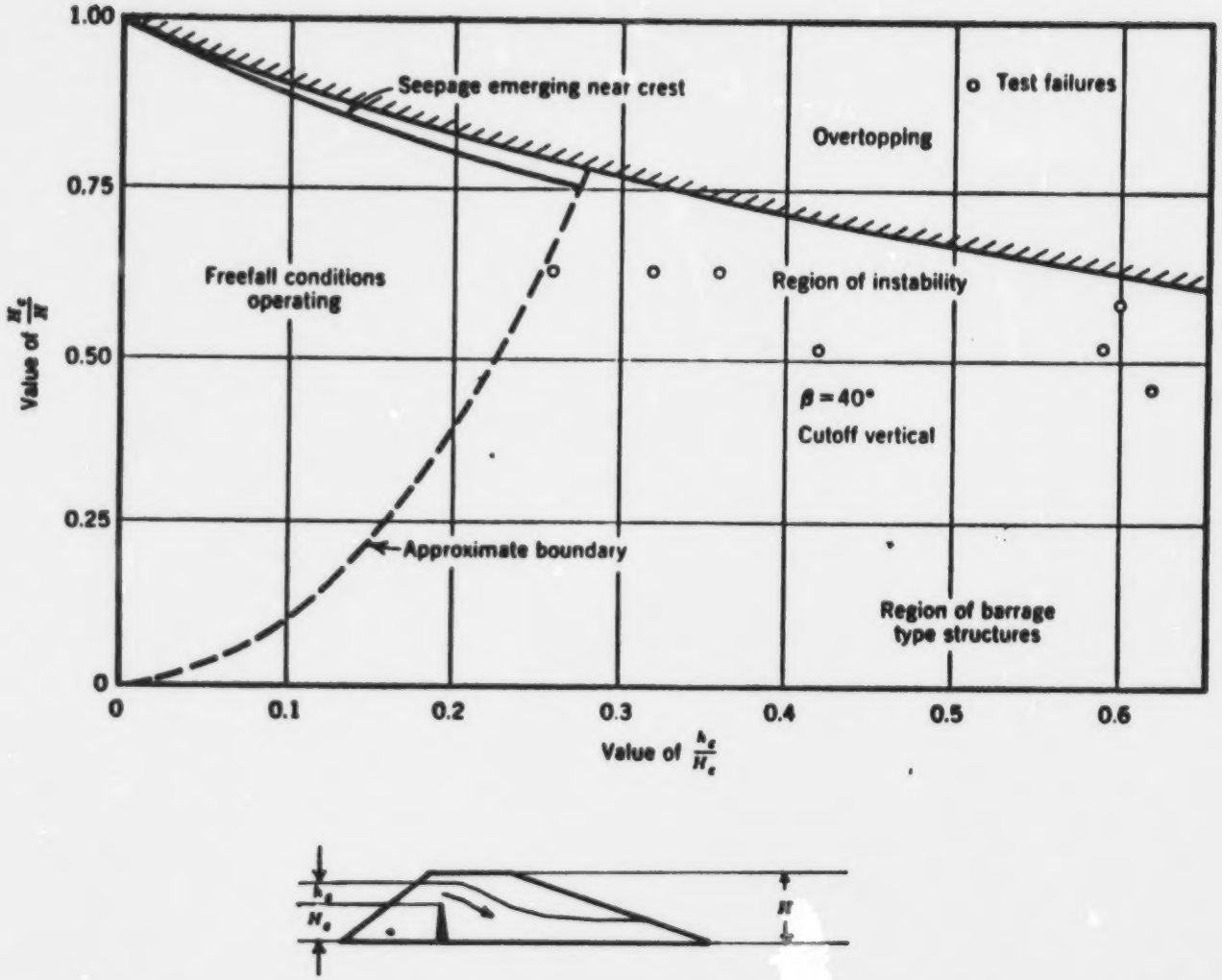
Figure 3 Mesh elements in Darcian and non-Darcian flow nets.




Regions of Flow in a Self-Spillway Dam

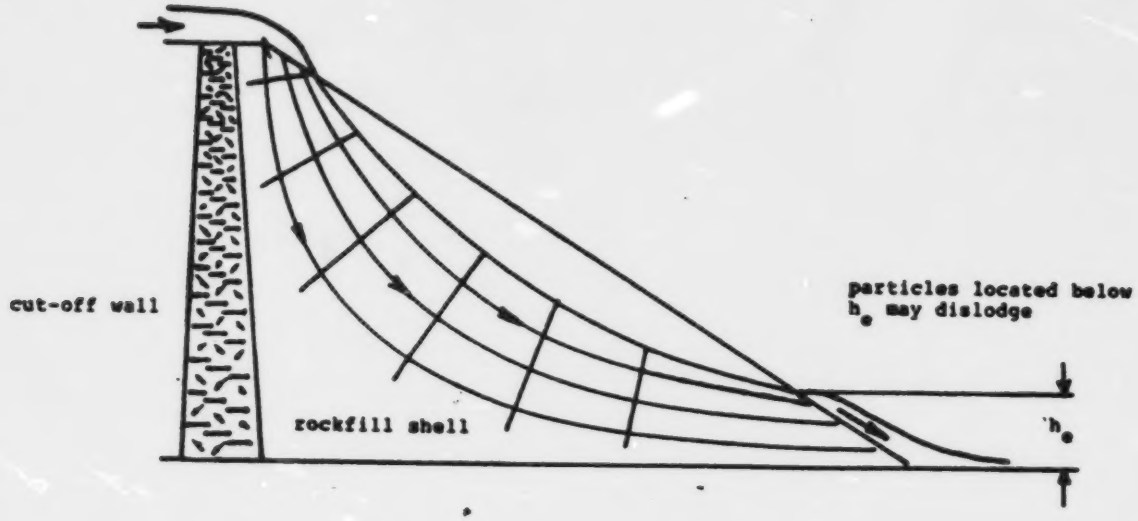
Source: Parkin, A.K., 1963. Rockfill dams with inbuilt spillways, Part I, Hydraulic characteristics. Bulletin 6, Univerisity of Melbourne and Water Research Foundation of Australia. March, 88 pp.

Figure 4(a) Zones of flow in flowthrough rockfill dam with an internal spillway.

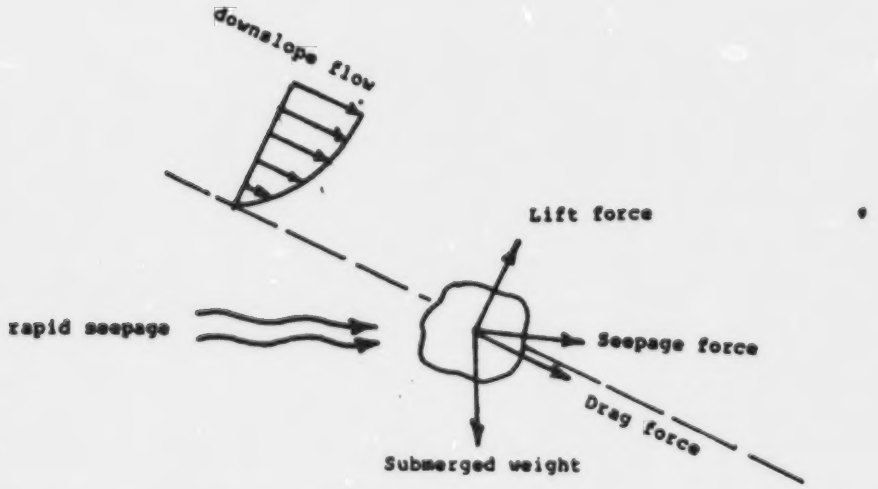



Source: Parkin, A.K., Trollope, D.H., and Lawson, J.D. 1966. Rockfill structures subject to water flow. Journal of the Soil Mechanics & Foundations Division, vol. 92, SM6, Nov., p. 135-151.

Figure 4(b) Stability of a rockfill dam with an inbuilt spillway.

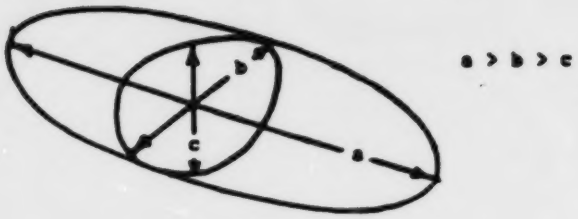



(a) Flowthrough seepage in a rockfill dam with a cut-off wall extending to the crest.

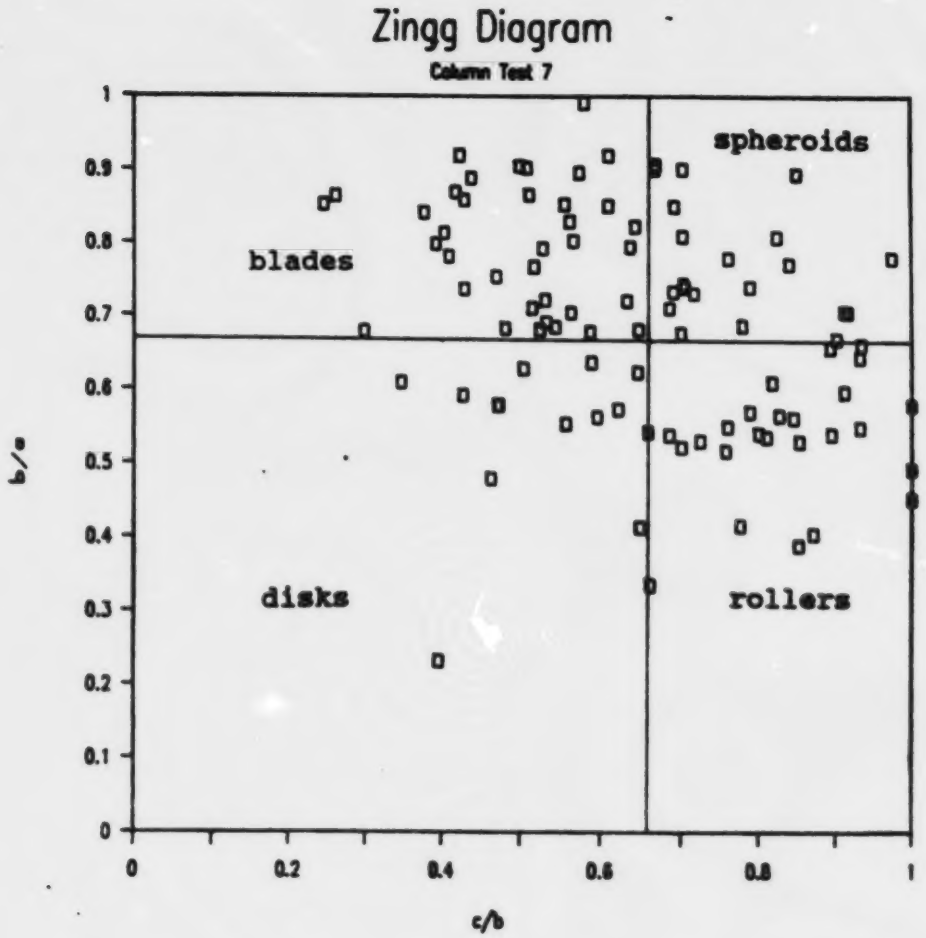



(b) Forces acting on a particle on the downstream face of a flowthrough rockfill dam.

Figure 5 Some factors pertaining to hydraulic stability of flowthrough and overflow rockfill dams.

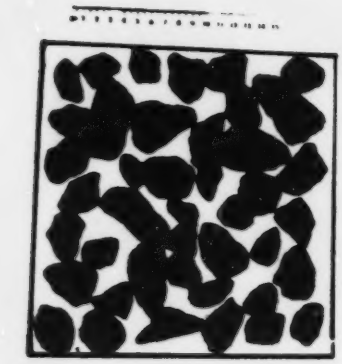



(a) Definition of particle axes.

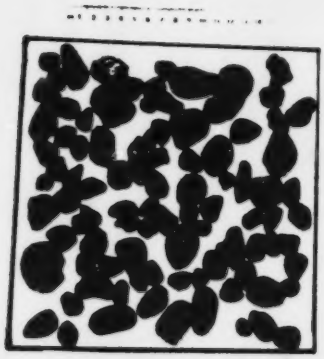



(b) Zingg diagram for material used in column test 7.

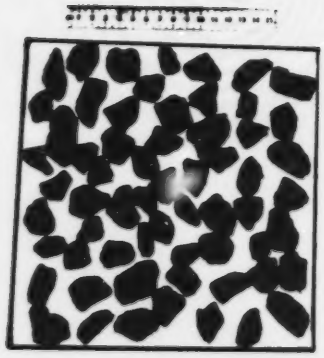
Figure 6 Characterisation of particle shapes.



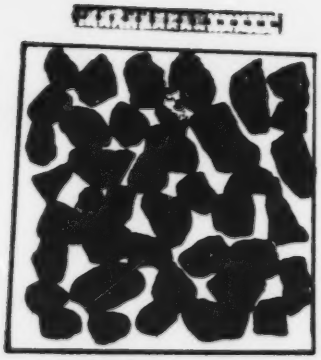
COLUMN TESTS 1 to 4



COLUMN TEST 5



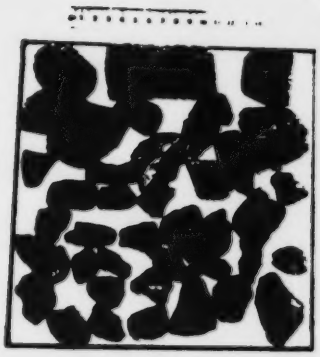
COLUMN TESTS 6, 11, 12



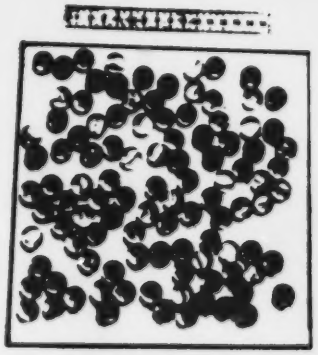
COLUMN TEST 7

Figure 7 Material used in column tests.

COLUMN TEST 8

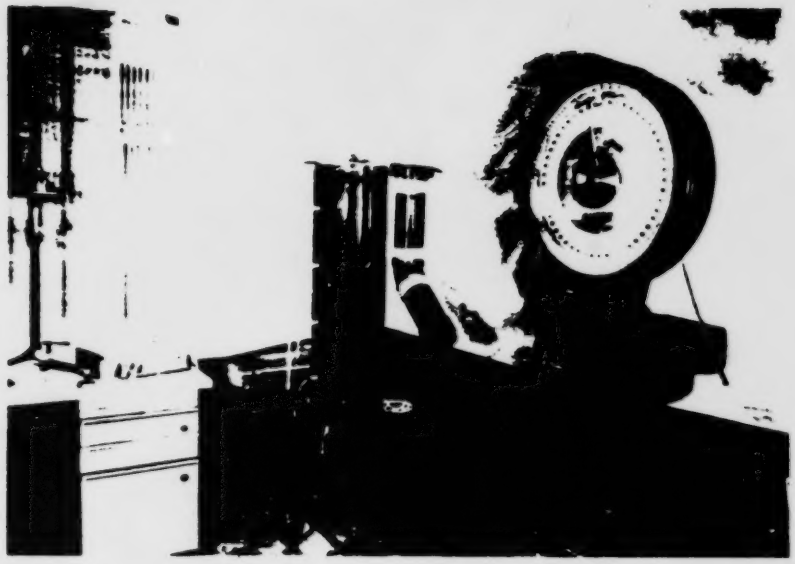


COLUMN TEST 9



COLUMN TEST 10

Figure 8 Material used in column tests.



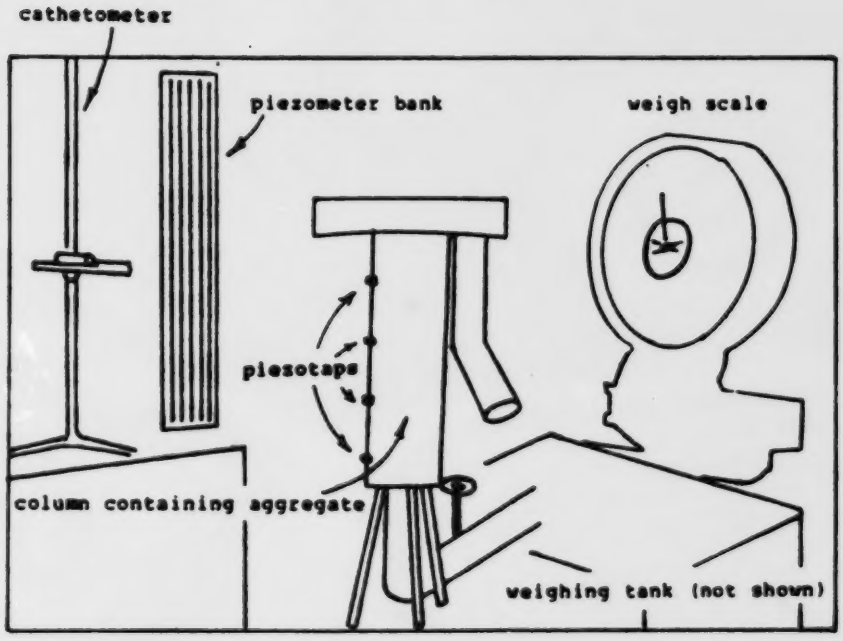
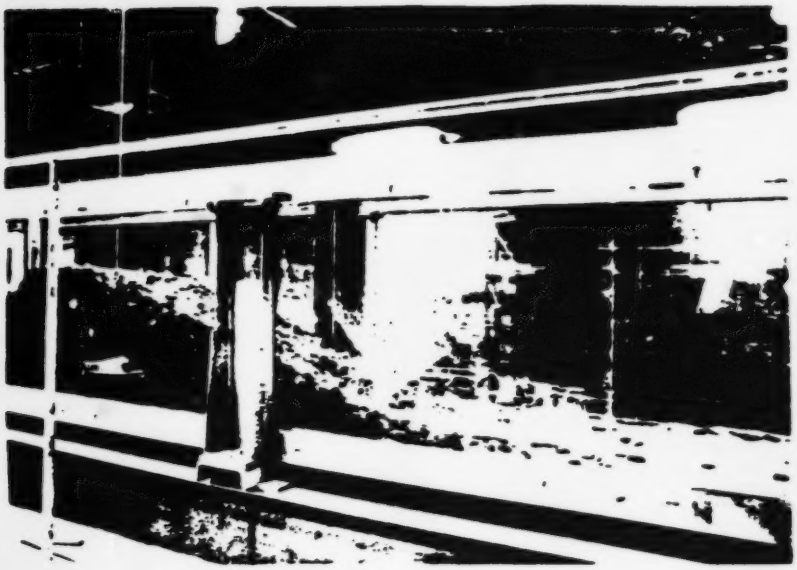



Figure 9 Packed column apparatus.



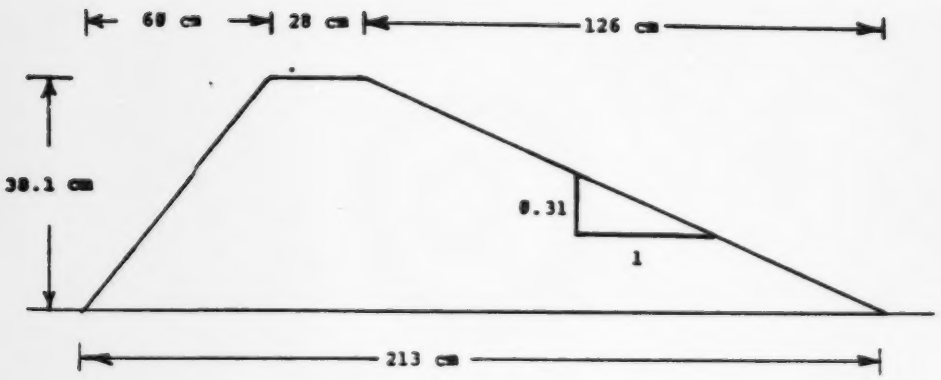



Figure 10 Flume with model dam.

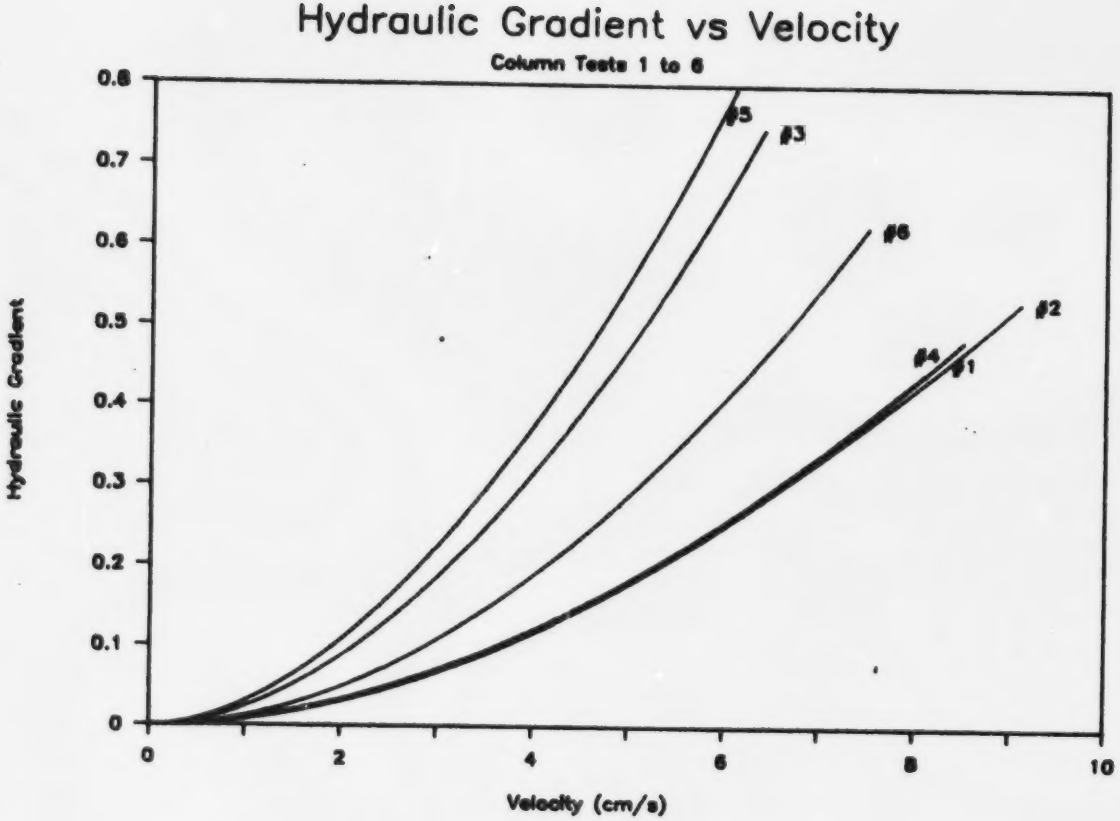



Figure 11 Experimentally determined curves relating hydraulic gradient and specific discharge, column tests 1-6.

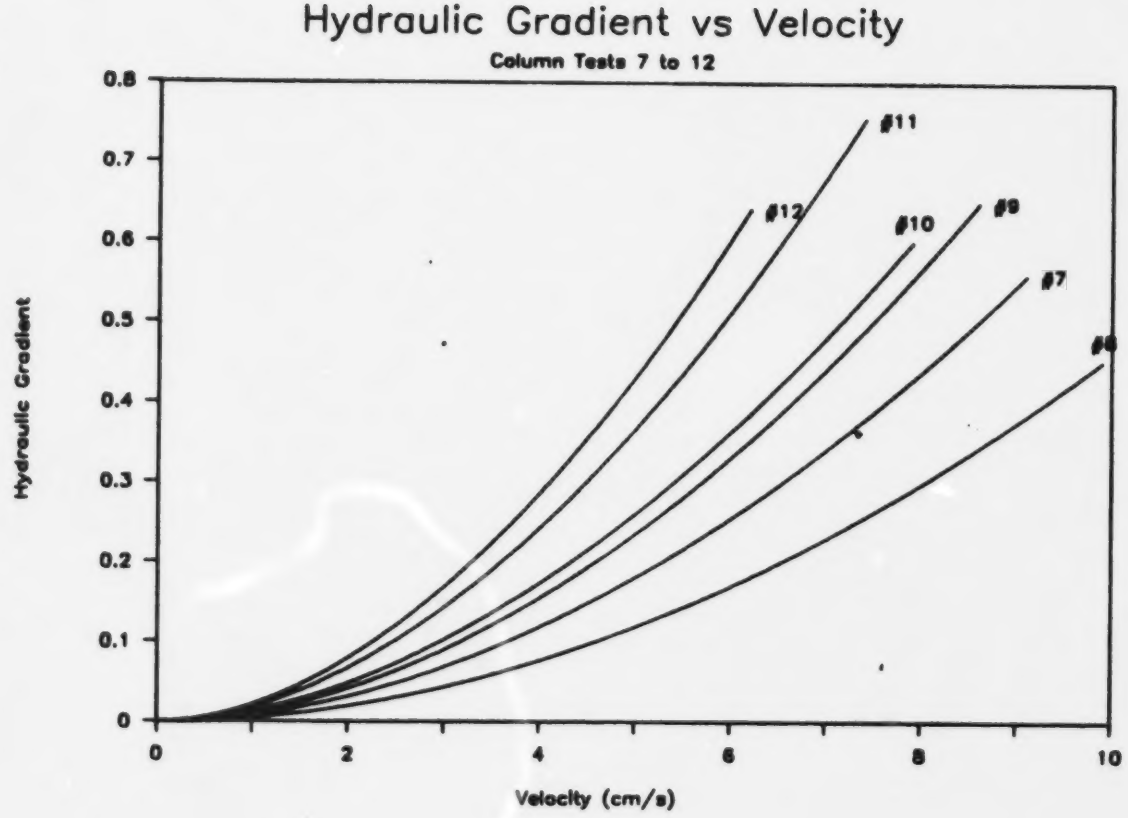



Figure 12 Experimentally determined curves relating hydraulic gradient and specific discharge, column tests 7-12.

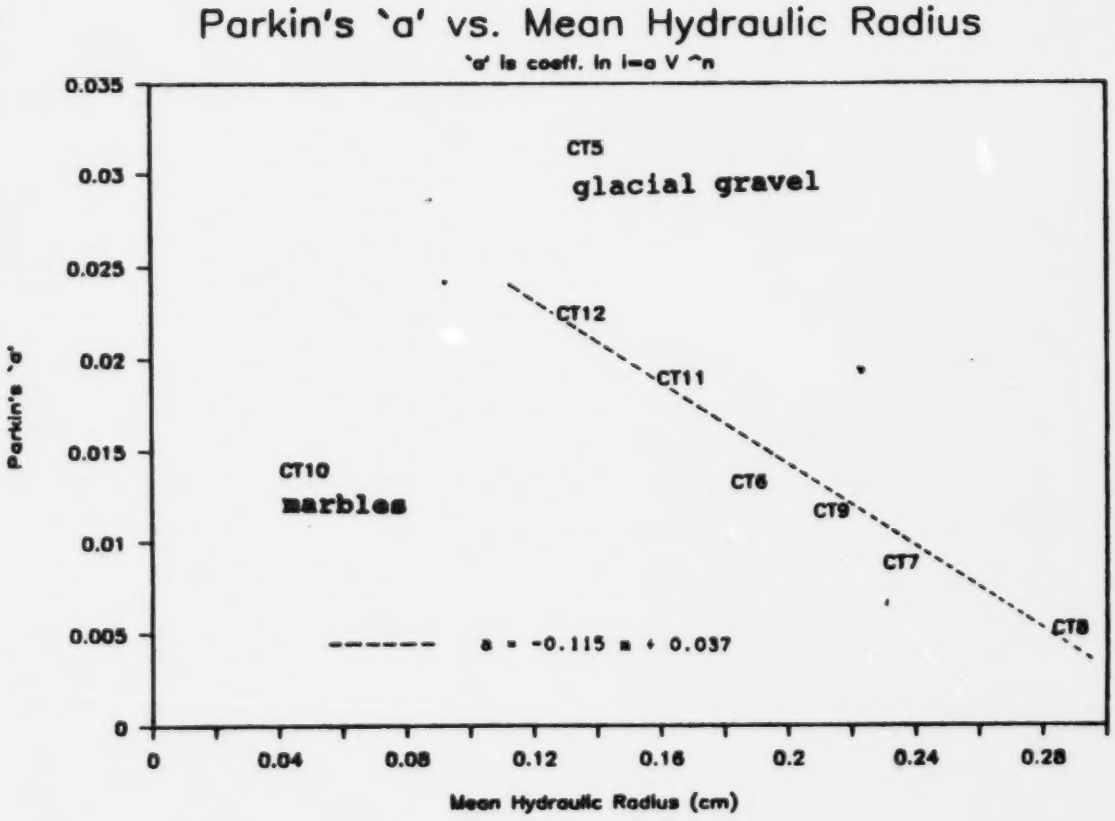
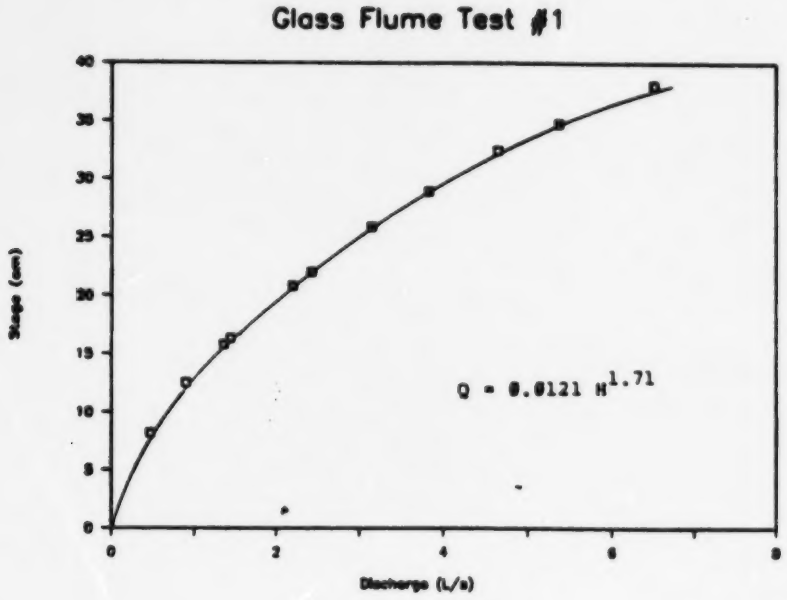
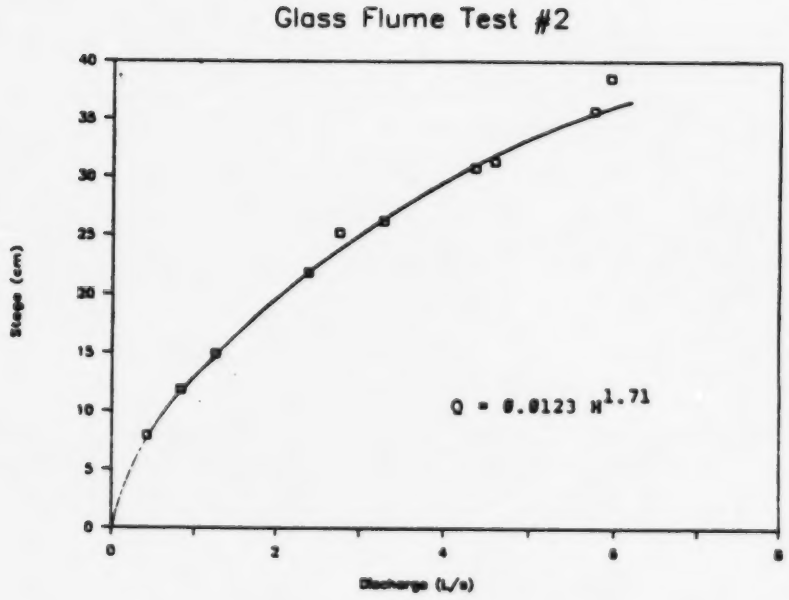



Figure 13 Coefficient 'a' versus mean hydraulic radius.




(a) Glass flume test 1.




(b) Glass flume test 2.

Figure 14 Stage-discharge curves for model flowthrough rockfill embankment having no cut-off wall.

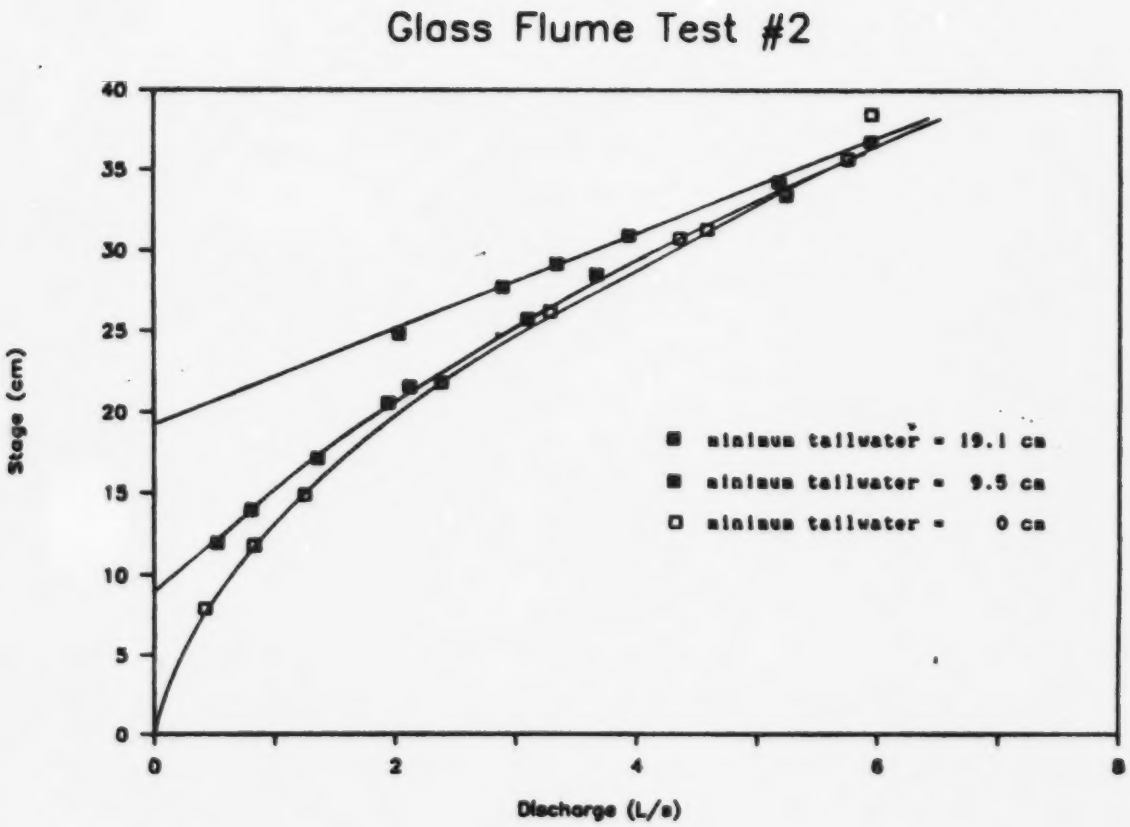
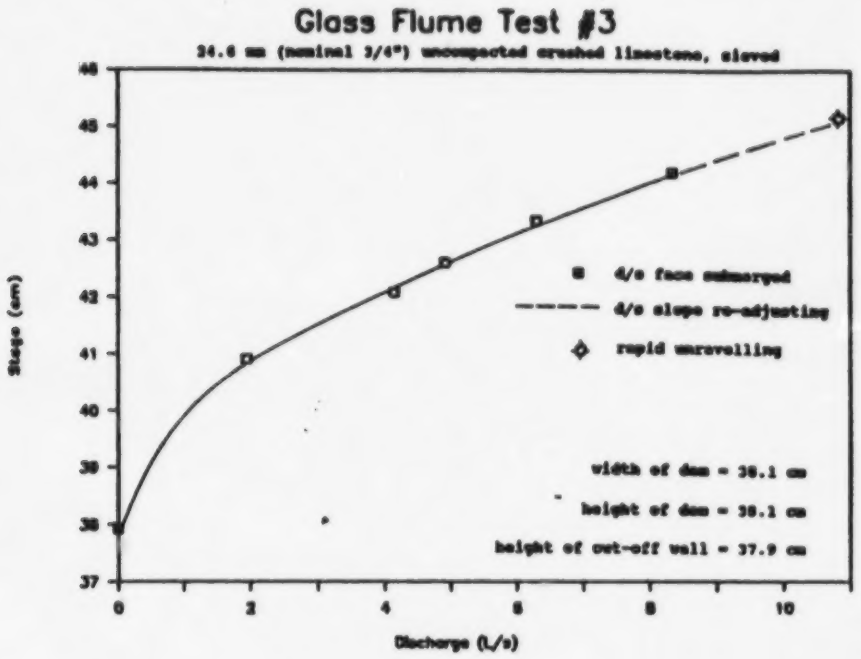
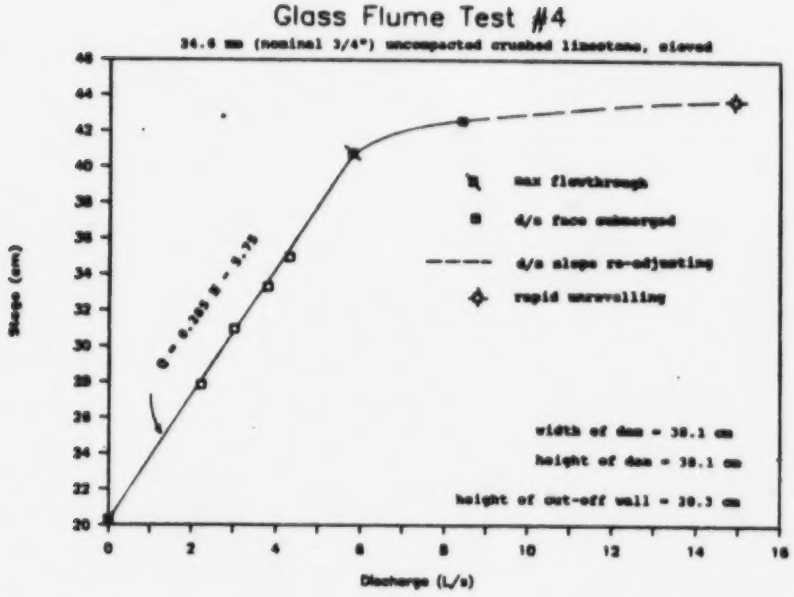



Figure 15 The effect of increasing tailwater elevation on stage-discharge curves for model flowthrough rockfill dam.


Glass Flume Test #3
34.6 mm (nominal 3/4") uncompacted crushed limestone, sieved
Stage (cm)
Discharge (L/s)
d/s face submerged
d/s slope re-adjusting
rapid unravelling
width of dam = 38.1 cm
height of dam = 38.1 cm
height of cut-off wall = 37.9 cm

(a) Stage-discharge curve for overflow dam.


Glass Flume Test #4
34.6 mm (nominal 3/4") uncompacted crushed limestone, sieved
Stage (cm)
Discharge (L/s)
max flowthrough
d/s face submerged
d/s slope re-adjusting
rapid unravelling
width of dam = 38.1 cm
height of dam = 38.1 cm
height of cut-off wall = 38.3 cm

(b) Stage-discharge curve for flowthrough dam.

Figure 16 Behaviour of model dams in overflow and throughflow modes of operation.

# APPENDIX II

## Table 1

Summary of Characteristics of Material Used for Column Tests. Column Tests 1 to 12, Part I.

| CT | Number of Particles Analysed | Nominal Sieve Size (in) | Mean b-axis (mm) | 95 Percent Confidence Interval expressed as percent of Mean | Sample Std Dev b-axis (mm) |
|---|---|---|---|---|---|
| 1 | 200 | 1" clear * | 25.2 | ± 2.6 % | 4.7 |
| 2 | 200 | 1" clear | 25.2 | ± 2.6 % | 4.7 |
| 3 | 200 | 1" clear | 25.2 | ± 2.6 % | 4.7 |
| 4 | 200 | 1" clear | 25.2 | ± 2.6 % | 4.7 |
| 4 B | 200 | 1" clear | 25.2 | ± 2.6 % | 4.7 |
| 5 | 100 | 3/4" minus | 16.1 | ± 5.6 % | 4.5 |
| 6 | 25 | 1/2" sieved | 19.4 | ± 6.8 % | 3.2 |
| 7 | 100 | 3/4" sieved | 24.6 | ± 2.7 % | 3.3 |
| 8 | 25 | 1" sieved | 35.9 | ± 5.3 % | 4.6 |
| 9 | ** NA | mixture | 26.6 | ± 6.8 % | 3.7 |
| 10 | 20 | not applic. | 15.6 | ± 0.9 % | 0.3 |
| 11 | 25 | 1/2" sieved | 19.4 | ± 6.8 % | 3.2 |
| 12 | 25 | 1/2" sieved | 19.4 | ± 6.8 % | 3.2 |

CT : column test.
* term used by crushed rock supplier for this material. Used as is.
** parameters inferred from tests 6, 7, and 8 in some cases.

Table 2

Summary of Characteristics of Material Used for Column Tests. Column Tests 1 to 12, Part II.

| CT | Mean a-axis (mm) | Mean c-axis (mm) | Porosity ( % ) | Coeff. Uniformity D60/D10 | Percent of Most Common Particle | Most Common Particle | Density (g/cm^3) |
|---|---|---|---|---|---|---|---|
| 1 | 39.0 | 15.0 | 53.3 | 1.30 | 34 | Disk | 2.695 |
| 2 | 39.0 | 15.0 | 54.1 | 1.30 | 34 | Disk | 2.695 |
| 3 | 39.0 | 15.0 | 46.7 | 1.30 | 34 | Disk | 2.695 |
| 4 | 39.0 | 15.0 | 47.0 | 1.30 | 34 | Disk | 2.695 |
| 4B | 39.0 | 15.0 | 46.6 | 1.30 | 34 | Disk | 2.695 |
| 5 | 22.6 | 10.9 | 40.1 | 1.60 | 34 | Spheroid | 2.653 |
| 6 | 33.5 | 11.6 | 48.1 | 1.50 | 32 | Disk | 2.695 |
| 7 | 36.7 | 15.7 | 47.4 | 1.24 | 39 | Disk | 2.695 |
| 8 | 54.3 | 23.1 | 46.9 | 1.28 | 36 | Disk | 2.695 |
| 9 | 41.5 | 16.8 | 45.7 | 1.62 | 36 | Disk | 2.695 |
| 10 | 15.6 | 15.6 | 40.0 | * 1.00 | 100 | Sphere | 2.650 |
| 11 | 33.5 | 11.6 | 45.1 | 1.50 | 32 | Disk | 2.695 |
| 12 | 33.5 | 11.6 | 40.0 | 1.50 | 32 | Disk | 2.695 |

CT : column test.
* top layer of marbles slightly smaller (= 11 % of total).

Table 3

Summary of Characteristics of Material Used for Column Tests. Column Tests 1 to 12, Part III.

| CT | Mean Particle Shape Factor | Mean Particle Spher-icity | Surface Area per unit Mass (cm^2/g) | Mass of Material (kg) | Mean Hydraulic Radius (cm) | Krumbein Round-ness Index |
|---|---|---|---|---|---|---|
| 1 | 0.49 | 0.64 | 1.151 | 84.37 | 0.368 | 0.2 |
| 2 | 0.49 | 0.64 | 1.151 | 82.95 | 0.380 | 0.2 |
| 3 | 0.49 | 0.64 | 1.151 | 96.21 | 0.283 | 0.2 |
| 4 | 0.49 | 0.64 | 1.151 | 95.74 | 0.286 | 0.2 |
| 4B | 0.49 | 0.64 | 1.151 | 96.44 | 0.281 | 0.2 |
| 5 | 0.57 | 0.70 | 1.843 | 106.57 | 0.137 | 0.8 |
| 6 | 0.45 | 0.59 | 1.831 | 93.80 | 0.188 | 0.2 |
| 7 | 0.52 | 0.66 | 1.424 | 94.92 | 0.235 | 0.2 |
| 8 | 0.52 | 0.65 | 1.135 | 95.98 | 0.288 | 0.2 |
| 9 | 0.50 | 0.63 | 1.463 | 98.08 | 0.213 | 0.2 |
| 10 | 1.00 | 1.00 | 5.253 | 106.63 | 0.048 | 1.0 |
| 11 | 0.45 | 0.59 | 1.831 | 99.14 | 0.167 | 0.2 |
| 12 | 0.45 | 0.59 | 1.831 | 108.36 | 0.135 | 0.2 |

CT : column test.

Table 4

Materials and Treatments for Column and Flume Test Porous Media

| Column Test | Material and Treatment |
|---|---|
| 1 | graded crushed limestone with a mean diameter of 25.2 mm, known as '1" clear' at the quarry. |
| 2 | as for #1. |
| 3 | as for #1 except material was compacted. |
| 4 | as for #1. |
| 5 | graded subrounded glacial gravel, 16.1 mm median diameter. |
| 6 | sieved crushed limestone, diameter of 19.4 mm, (nominal 1/2"). |
| 7 | sieved crushed limestone, diameter of 24.6 mm, (nominal 3/4"). |
| 8 | sieved crushed limestone, diameter of 37.9 mm, (nominal 1"). |
| 9 | mixture of equal parts of crushed rock used for tests 6, 7, and 8. |
| 10 | 15.75 mm glass spheres. |
| 11 | sieved crushed limestone, diameter of 19.4 mm, (nominal 1/2") slightly compacted. |
| 12 | as for 11, except heavily compacted. |

| Glass Flume Test | |
|---|---|
| 1 | '1" clear' uncompacted crushed limestone, as for CT1 |
| 2 | '1" clear' uncompacted crushed limestone, as for CT1 |
| 3 | nominal 3/4" sieved uncompacted crushed limestone, as for CT7 |
| 4 | nominal 3/4" sieved uncompacted crushed limestone, as for CT7 |

Table 5

Summary of Parameters for Equations Developed.

Form: $i = a\ v^{N}$

| Column Test | Value of 'a' | Value of 'N' |
|---|---|---|
| CT 1 | 0.0085 | 1.89 |
| CT 2 | 0.0100 | 1.80 |
| CT 3 | 0.0248 | 1.83 |
| CT 4 | 0.0094 | 1.84 |
| CT 4B | 0.0107 | 1.87 |
| CT 5 | 0.0314 | 1.79 |
| CT 6 | 0.0133 | 1.91 |
| CT 7 | 0.0088 | 1.88 |
| CT 8 | 0.0053 | 1.94 |
| CT 9 | 0.0117 | 1.87 |
| CT 10 | 0.0140 | 1.82 |
| CT 11 | 0.0190 | 1.84 |
| CT 12 | 0.0225 | 1.84 |
| means: | 0.0146 | 1.86 (cm-sec units) |

# APPENDIX III

Grain size curves for aggregates used in column and flume tests.

1. Grain size curve for crushed limestone used in column tests 1-4 and flume tests 1 and 2.

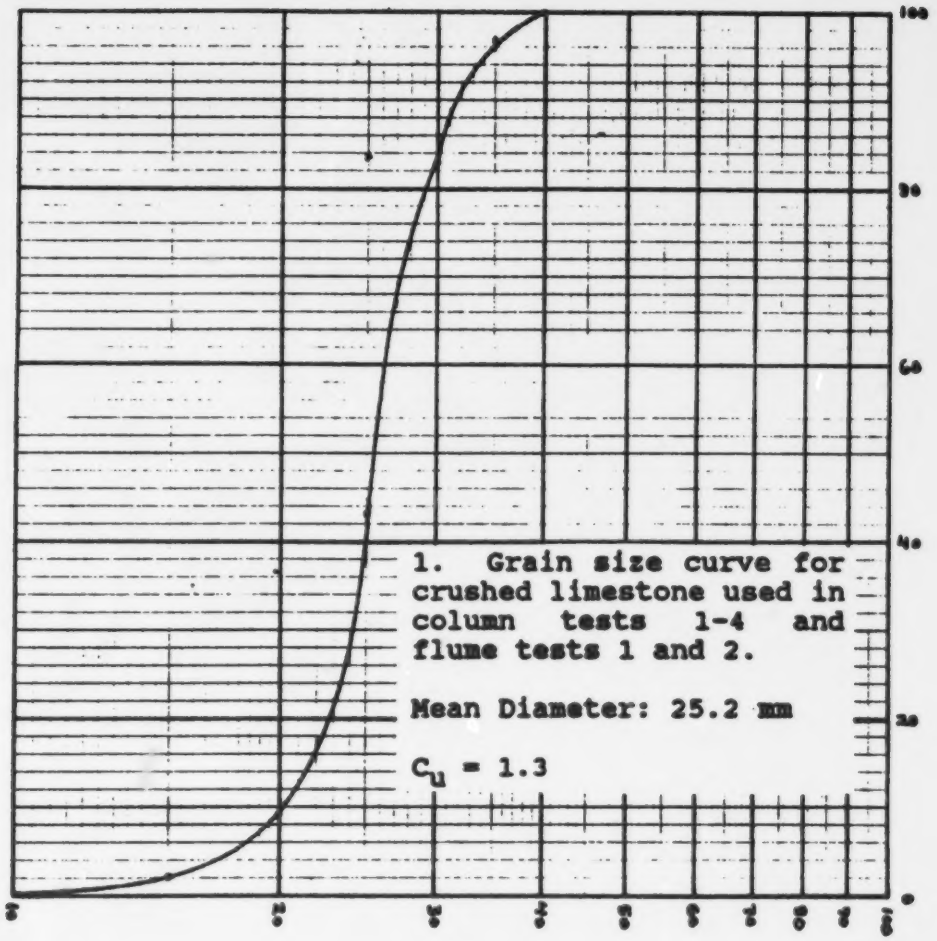
2. Grain size curve for graded glacial gravel used in column test 5.

3. Grain size curve for crushed limestone used in column tests 6, 11, and 12.

4. Grain size curve for crushed limestone used in column tests 7 and flume tests 3 and 4.

5. Grain size curves for crushed limestone used in column tests 8.

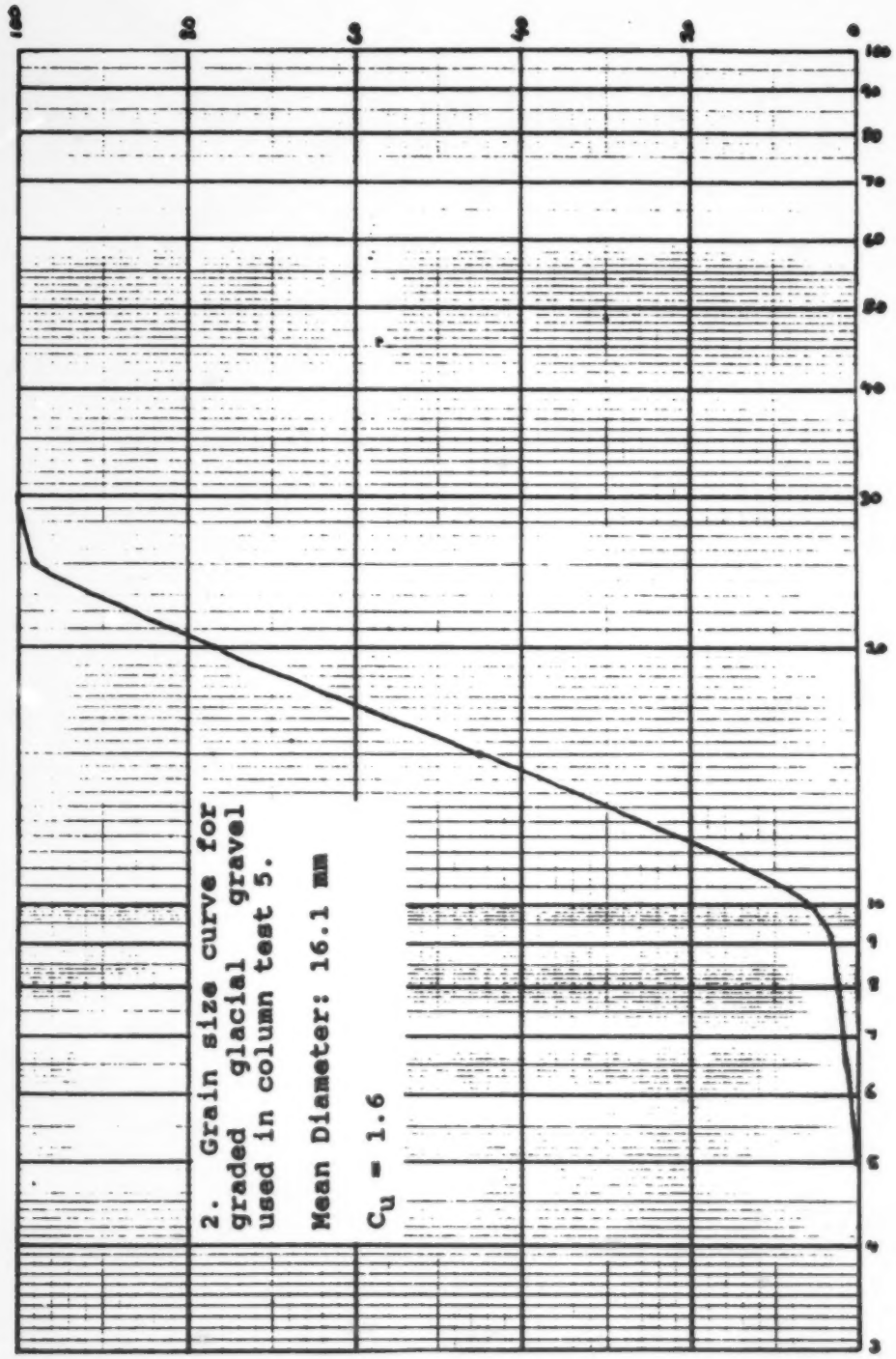
6. Grain size curves for crushed limestone used in column tests 9.

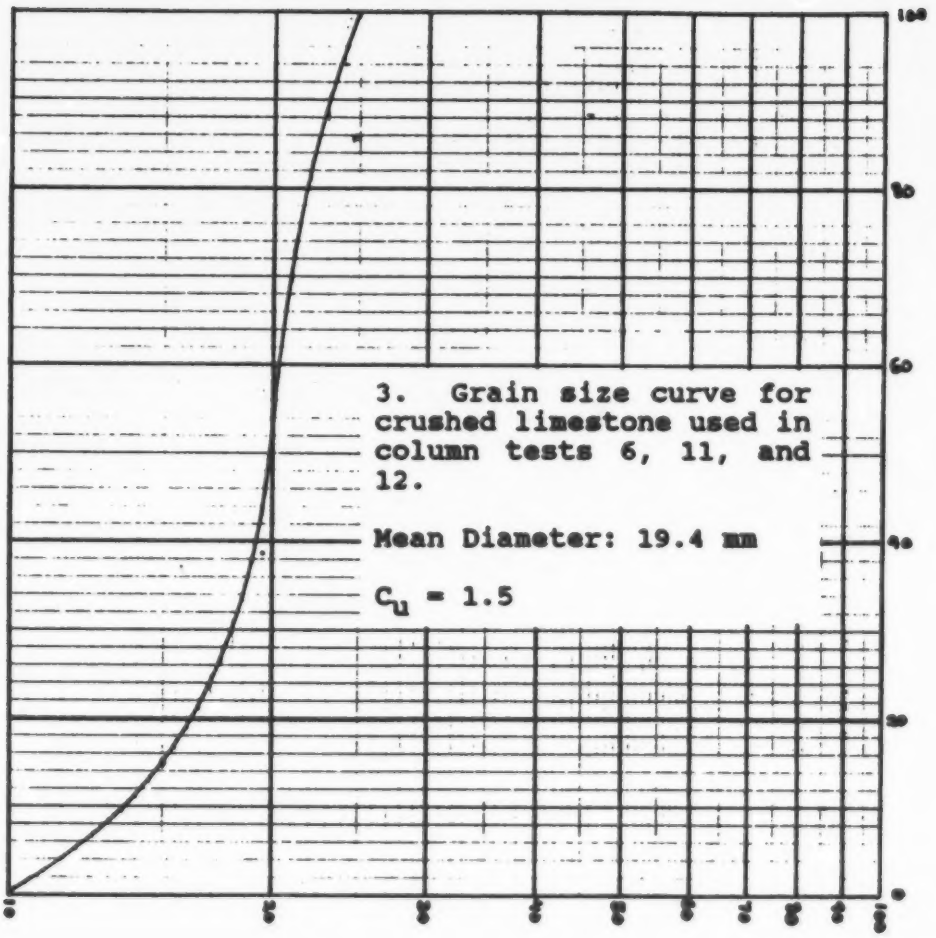



1. Grain size curve for crushed limestone used in column tests 1-4 and flume tests 1 and 2.

Mean Diameter: 25.2 mm

$C_u$ = 1.3

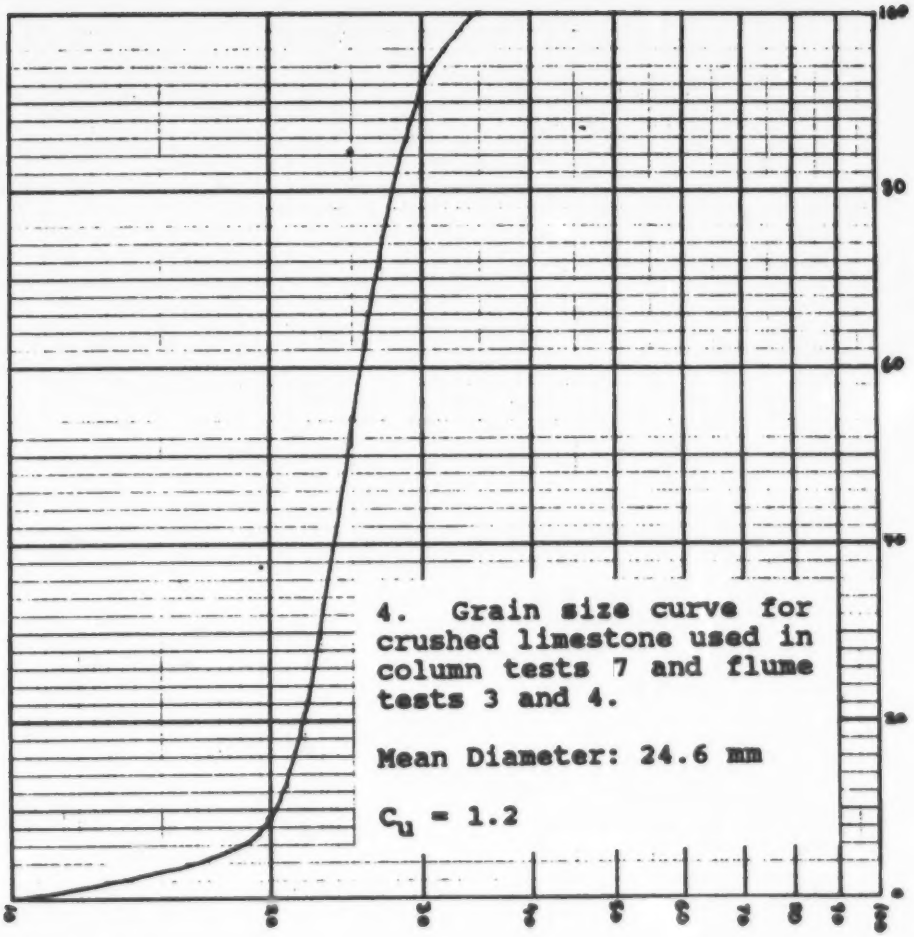

Percent Passing
Length of b-axis (mm)
2. Grain size curve for graded glacial gravel used in column test 5.
Mean Diameter: 16.1 mm
$C_u$ = 1.6

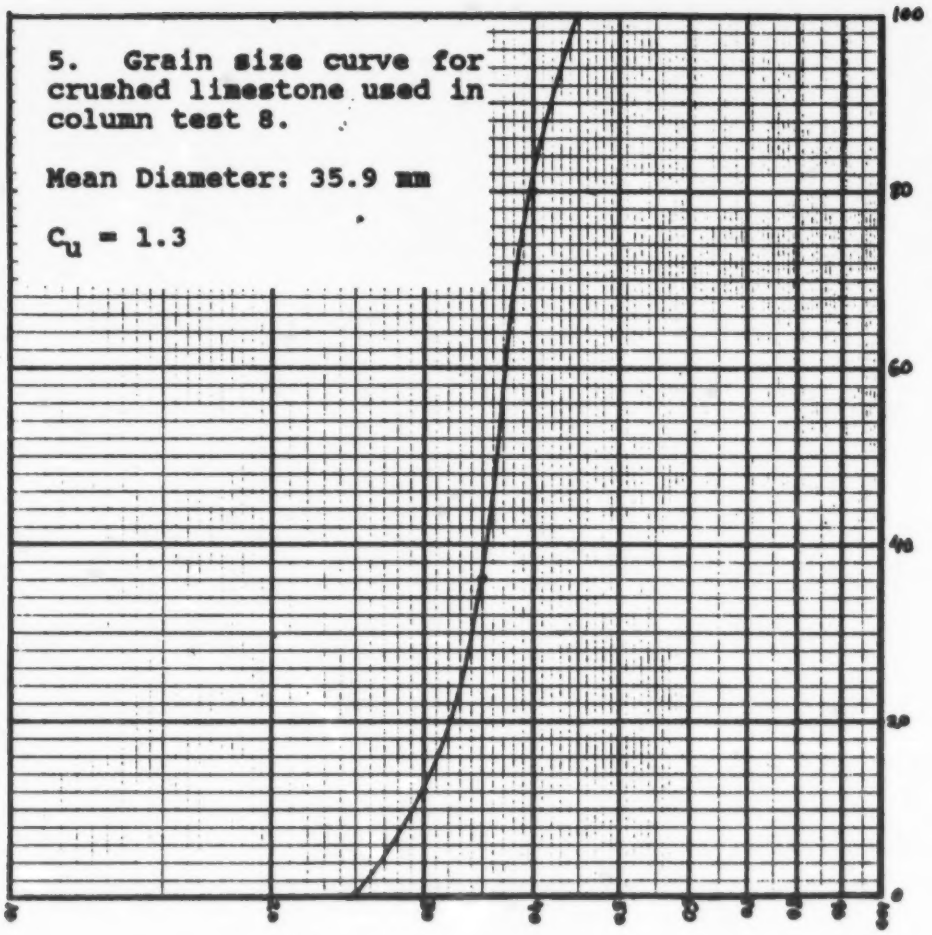


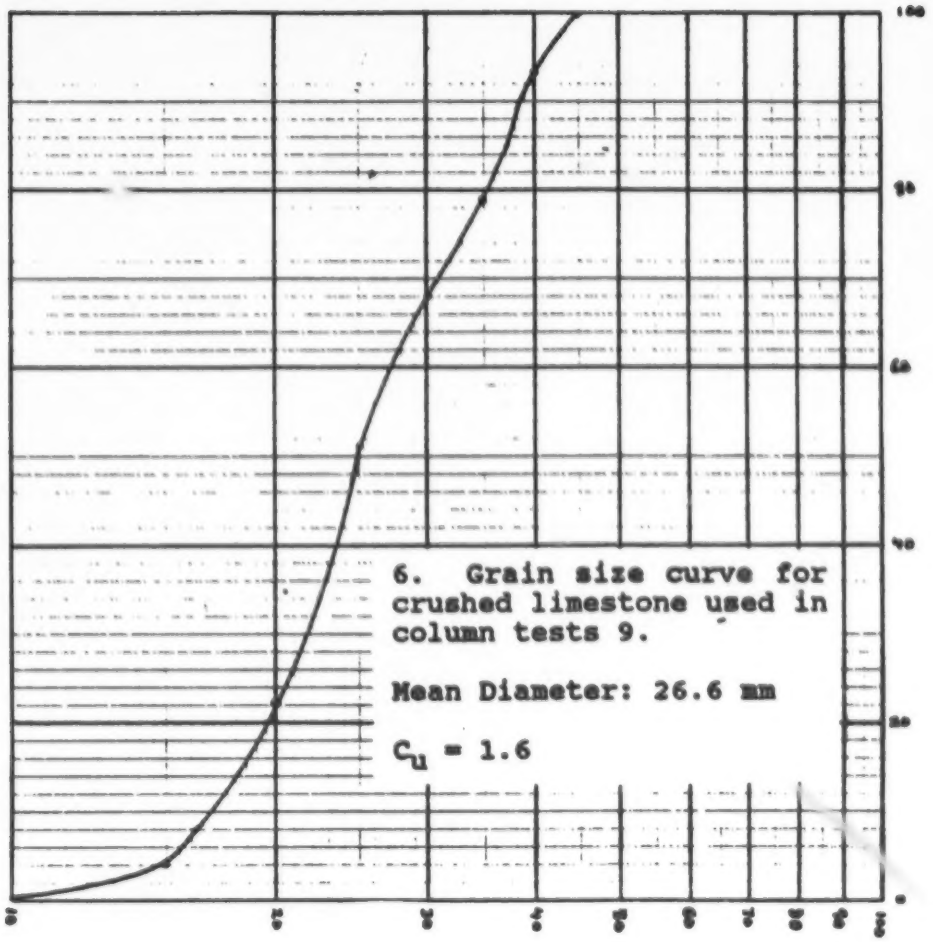

3. Grain size curve for crushed limestone used in column tests 6, 11, and 12.

Mean Diameter: 19.4 mm

$C_u$ = 1.5


Percent Passing
Length of b-axis (mm)
4. Grain size curve for crushed limestone used in column tests 7 and flume tests 3 and 4.
Mean Diameter: 24.6 mm
$C_u$ = 1.2


5. Grain size curve for crushed limestone used in column test 8.
Mean Diameter: 35.9 mm
$C_u$ = 1.3
100
80
60
40
20
0
Percent Passing
Length of b-axis (mm)




6. Grain size curve for crushed limestone used in column tests 9.

Mean Diameter: 26.6 mm

$C_u$ = 1.6

## APPENDIX IV

### OUTLINE OF WORK REQUIRED FOR PHASE II

Item 1. Further work on rapid methods of obtaining the surface area per unit volume (or mass) for the aggregate studies.

Goal: To more rapidly determine effective pore diameter.

Tasks: develop a relationship between the volume given by the enclosing ellipsoid of a particle and the surface area of the particle.

Item 2. Further studies on flow through packed columns.

Goals:
To substantiate the relationship given by equation [18], relating the value of 'a' to the mean hydraulic radius for crushed stone.

Tasks:
- determine parameters in equation: $i = a V^N$.
- measure or estimate mean hydraulic radius for each material tested.

Item 3. Flume studies at the University of Ottawa.

Goal: to characterise flowthrough rockfill dams with a full height cut-off wall as a hydraulic structure interms of capacity and stability.

Tasks:
a) Experimental.
- measure pressure distribution within model dams.
- measure the free surface profile for each test.
- determine optimum placement of the cutoff wall.
- determine how seepage rates within the downstream shell affects the stage-discharge curve and the stability of particles near the toe of the dam.
- observe the gross stability and modes of failure of the model dams.

b) Analytical.
- develop theoretical stage-discharge curves for selected flowthrough dams with high inbuilt spillways, and compare with observations.
- sketch flownets using Wilkin's method, and compare with measured internal pressures.

- knowing 'a' and 'N' perform finite difference analysis to compute flownets using modified method of Curtis and Lawson.
- compute height of maximum point of emergence on downstream face using Parkin's method and compare with results of the above two computational methods and with observations.

Item 4. Studies of flow under ice conditions.

a) Work in indoor ice tank at NRC.

Goals:
- to measure the quantity of flowthrough when an ice-cover exists upstream.
- to monitor the formation of ice in the voids of the structure under controlled heat loss conditions, and to observe how flowthrough behavior is affected by these phenomena.

b) Near-prototype scale studies in the NRC outdoor facility.

Goal: - to document when and under what conditions ice blockage of the voids occurs with typically variable winter conditions. Data required: meteorological data, continuous flowthrough data, measurement of ice buildup in rockfill

Item 5. Field studies to observe the behaviour of flowthrough dams and rock drains in Canada. Cooperation with the owners will be essential in order to undertake this important phase of the work. The assistance of Energy Mines and Resources Canada will be used to establish contacts with the owners of such facilities.